GROUND-WATER CONTAMINATION ASSESSMENTS

By

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PREFACE

Throughout the past ten years, I have spent my working days investigating ground-water contamination incidents caused by industrial activities, evaluating and explaining their environmental consequences to non-specialists and specialists alike, and developing a research program for the U.S. Environmental Protection Agency on the related topics of hydrologic processes and mathematical modeling of ground-water contaminant transport. In my contacts with other scientists, engineers, water-system managers, attorneys, policymakers, and concerned citizens it has always been clear to me that public health and safety demand the most careful assessment of society's ground-water contamination problems, and the most judicious implementation of solutions to those problems.

My experiences have led me to believe that scientific theories have far outpaced their practical application as a result of continuing difficulties in obtaining reliable field data upon which to base decisions, and that this fact is not well explained to, nor well appreciated by, decisionmakers. I also believe that the gap in effective communication between specialists and non-specialists is not narrowing, that much technical work is not properly focused because of this gap, and that this often causes decisions to be made that fall short of what is economically and socially desireable. This work is an attempt to express those beliefs in concrete terms, in the hope that it will help to catalyze greater efforts by all parties to understand the real capabilities and limitations associated with solving ground-water contamination problems.

Of the many people instrumental in shaping my career and helping me to complete this work, I want to first recognize my best friend and constant companion of the past 15 years, my lovely wife Rosemary Church Keely. A more loving, noble spirit I have yet to encounter; she has rescued me from my most despairing moments in the long trek to this point in my professional career.

Dr. Don F. Kincannon is a wonderful human being from whom I have gained far more than technical guidance; I hope to emulate his professionalism and common-sense approach to difficult problems. Dr. Richard N. DeVries deserves special mention as a treasured confidant; his concern and advice helped me through a difficult time early in this work. My sincere gratitude goes to the other members of my thesis committee as well; Dr. Dougas C. Kent, Dr. Allen E. Kelly, and Dr. William F. McTernan all gave their valuable time and advice without hesitation.

To the managers, scientists, and support staff at EPA's Robert S. Kerr Environmental Research Laboratory in Ada, Oklahoma, where most of the work described herein originated, my heartfelt thanks for the friendship, the improved understanding of subsurface contaminant transport processes, and the financial, technical, and moral support. Although space limitations prohibit citing each of you by name, you and I know what we have shared and will remember these days with fondness.

To those professional acquaintances, present and past colleagues, and other individuals that I have forgotten to acknowledge in my haste, thank you.

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LIST OF SYMBOLS

A	cross-sectional area perpendicular to flow (L^2)
å	angle in cartesian coordinate system (degrees)
a	dispersivity along the x-axis (L)
b	dispersivity along the y-axis (L)
c	dispersivity along the z-axis (L)
C	concentration at existing state (M/L)
C_e	concentration at chemical equilibrium (M/L)
D_x	coefficient of dispersion along the x-axis (L^2/t)
D_y	coefficient of dispersion along the y-axis (L^2/t)
D_z	coefficient of dispersion along the z-axis (L^2/t)
$D^{\bullet}$	coefficient of molecular diffusion (L^2/t)
d/dt	ordinary first derivative with respect to time coordinate
d/dx	ordinary first derivative with respect to x-coordinate
d/dy	ordinary first derivative with respect to y-coordinate
d/dz	ordinary first derivative with respect to z-coordinate
g	acceleration due to gravity (L/t^2)
h	height of an imaginary cylinder in radial well flow, distance of draw, and capture zone discussions (L)
I	hydraulic gradient (dimensionless)
K	hydraulic conductivity (L/t)

k	intrinsic permeability (L^2)
K_d	partition coefficient derived from linearization of Freundlich isotherm (L^3/M)
K_F	Freundlich sorption isotherm constant (L^3/M)
N	number of atoms of parent radionuclide at any time
$1/n$	Freundlich sorption isotherm exponent (dimensionless)
p_b	density of bulk aquifer matrix (M/L^3)
p_f	density of aquifer fluids (M/L^3)
p_s	density of aquifer solids (M/L^3)
Q	flowrate (L^3/t)
R	retardation factor, equal to ratio of average seepage velocity to apparent average velocity of contaminant (dimensionless)
r	radial distance coordinate (L)
r_c	internal radius of well casing (L)
r_p	external radius of pump column (L)
r_{sp}	radial distance to the stagnation point of pumping wells (L)
r_1	length of small radius of an ellipse (L)
r_2	length of large radius of an ellipse (L)
S	coefficient of storage of an aquifer (dimensionless)
s	water level drawdown or decline in piezometric surface (L)
Δs	change in drawdown over one log-cycle of time, used in the Cooper-Jacob approximation to the Theis solution (L)
T	transmissivity of an aquifer (L^2/t)

t	time coordinate (t)
t_c	time that waters in casing storage contribute to pumpage (t)
t_o	time value found by extrapolating a Cooper-Jacob plot back to zero drawdown for storage coefficient estimates (t)
V	average seepage velocity in cartesian coordinate system (L/t)
V_c	apparent average velocity of contaminant movement (L/t)
V_n	seepage velocity of the natural system in the absence of pumpage influences (L/t)
V_p	seepage velocity caused by a pumping well under radial flow assumptions (L/t)
V_r	radial component of average seepage velocity in cylindrical coordinate system (L/t)
V_β	angular component of average seepage velocity in cylindrical coordinate system (L/t)
V_x	x-axis component of average seepage velocity (L/t)
V_y	y-axis component of average seepage velocity (L/t)
V_z	z-axis component of average seepage velocity (L/t)
V_D	Darcy 'flux' (L/t)
Vol_c	volume of cylinder (L^3)
Vol_e	volume of elliptically shaped layer (L^3)
Vol_r	volume of rectangular box (L^3)
w	width of a rectangular box (L)
W_{max}	maximum width of capture zone of a pumping well (L)
$W(u)$	well function of Theis equation solution

x/m	mass of contaminant sorbed to solids in Freundlich isotherm experimental analysis (dimensionless)
cos	geometric cosine function
sin	geometric sine function
tan	geometric tangent function
atan	geometric arctangent function
$\delta/\delta t$	partial first derivative with respect to time coordinate
$\delta/\delta x$	partial first derivative with respect to x-coordinate
$\delta/\delta y$	partial first derivative with respect to y-coordinate
$\delta/\delta z$	partial first derivative with respect to z-coordinate
μ	fluid viscosity (M/L/t)
π	numerical constant expressing ratio of diameter of a circle to its circumference (3.1415972...), 'pi'
Σ	summation notation, quantities to right of symbol are arithmetically summed and quantity is then acted upon
ϕ_e	effective porosity (dimensionless)

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L = length unit   M = mass unit   t = time unit

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CHAPTER I

INTRODUCTION

1.1 Ground-Water Contamination in the U.S.

The United States is the fourth largest country in the world, with total land surface of 9.4 million square kilometers. Its 217 million inhabitants live in a moderately low average population density (23 per sq.km.) and despite notable exceptions, such as New York City and Los Angeles, the average home is rural or small town. At least one-half of the U.S. population relies on ground water for drinking water, including 95 percent of all rural households, whose ground water is commonly used untreated.

Total ground-water usage as of 1980 was 1.3×10^{14} liters, with two-thirds of that applied to agriculture, one-sixth distributed for drinking water, and the remainder used by various industries (AIPG, 1983). Virtually every agricultural, mining, power, and manufacturing industry is active in the U.S.; satisfactory disposal of the millions of kilograms of solid and liquid wastes produced by them is a difficult issue and one which impacts ground-water quality management directly.

1.1.1 Ground-Water Quantity Laws

Compared to the other highly industrialized nations of the world, the United States is relatively young and much of its sociopolitical policies are still evolving. Nowhere is this more clear than in the field of water resources management. The system for allocation of water rights varies significantly among the States and involves the establishment of certain personal property rights by either the riparian or the appropriation doctrines, or by some combination of the two (Figure 1.1). Historically these doctrines were applied to the establishment of stream-water usage priorities, but rapid expansion of ground-water usage in the last century, together with an improved understanding of the interrelationship of streamflows and aquifer pumpage, has led to extension of these doctrines to ground-water allocation. An examination of these pre-existing laws and regulations governing water acquisition, ownership, and responsibility helps one put into proper perspective the recent laws passed to minimize ground-water contamination by waste disposal controls.

The riparian ('on the bank') doctrine requires direct physical proximity to a stream to obtain water rights. Frontage on the bank of the stream by some part of the property to which the water will be applied is necessary to establish and maintain a riparian right; as a consequence, a strictly riparian right cannot be bought or sold for use on other property. This doctrine originated in the humid eastern United States where the consumptive use of water for irrigation was far secondary to its use as a transportation medium; the overall effect of the riparian doctrine was to limit actual withdrawals to insignificant amounts to ensure that adequate flow continues downstream (Kazmann, 1972).

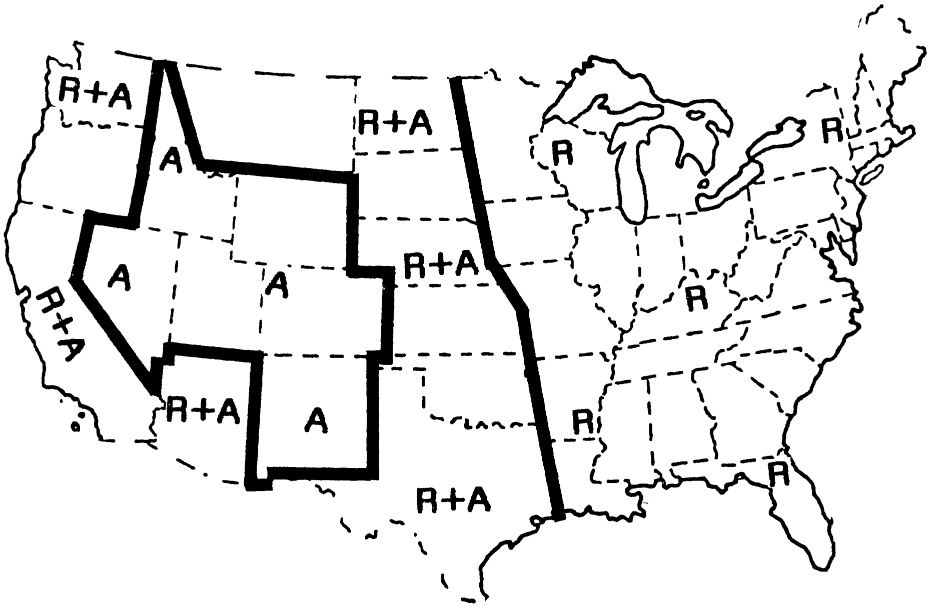
In theory, each riparian owner is entitled to the full flow of the stream undiminished in quantity or quality; but this goal is moderated by

RIPARIAN DOCTRINE

〉 "ON THE BANK" – LAND OWNERSHIP DERIVED
〉 'REASONABLE USE THEORY' VS. 'FULL FLOW OF STREAM UNDIMINISHED IN QUANTITY OR QUALITY'
〉 PROPORTION ALLOCATIONS IN TIMES OF SHORTAGES BY 'PRINCIPLE OF CORRELATIVE RIGHTS'
〉 DISCOURAGES CONSUMPTIVE USE

APPROPRIATION DOCTRINE

〉 "FIRST IN TIME, FIRST IN RIGHT" – BY FILING DATES
〉 FLOW RATE SPECIFIED IN PERMIT, WITH PRIORITY
〉 WATER MUST BE PUT TO BENEFICIAL USE
〉 WATER AND RIGHTS MAY BE SOLD AS PROPERTY



R = RIPARIAN A = APPROPRIATION R+A = BOTH

Figure 1.1. Prevalence of Riparian and Appropriation Doctrines

the principle of reasonable use which allows for practical diminishment of quantity and quality by each riparian owner. The amount of water normally allocated to that property by reasonable use is dependent on the size of the property contiguous to the river frontage. A corollary of reasonable use is the principle of correlative rights, which holds that all riparian owners share proportionately in the misfortunes of droughts or other water limiting events.

It is clear from what has been stated thus far that some conceptual difficulties accompany the application of the riparian doctrine to the allocation of ground-water rights. The major difficulty relates to the need to establish a basis for deciding what constitutes the full flow of the aquifer beneath a landowners property and how to proportion correlative rights in times of inadequate recharge. For this reason, most States in which the riparian doctrine historically predominates have had to create separate permitting systems to limit ground-water rights to specific flow rates. While such systems more closely follow principles of the appropriation doctrine than they do the riparian doctrine, they generally do not allow for the direct conjunctive management of stream and ground-water allocations that is possible under purely appropriation systems.

Under the appropriation ('first in time, first in right') doctrine, which predominates in the arid western United States, allocations of stream waters have been obtained based on the chronology of the permit filings and have not been tied to proximity of the property to the stream. Unlike the non-transferrable water rights obtainable under riparian doctrine, water rights obtained under the appropriation doctrine are specific property rights which may be bought and sold independent of the property to which they first applied.

An important facet of this kind of arrangement is that water rights under appropriation doctrine are issued for specific flow values, are exploitable only after satisfaction of all prior rights, and must be put to beneficial use continually or the rights are forfieted (Kazmann, 1972). Direct transference of these precepts of the appropriation doctrine of water rights to the allocation of specific ground-water rights poses no conceptual problem and also facilitates the conjunctive management of stream waters and ground waters.

This brief description of the riparian and appropriation doctrines does not introduce all of their details, but it should suffice to convey the impression that the ownership of fresh waters in the United States is subject to a mixture of sometimes conflicting regulations. The added importance of a brief understanding of the development of regulations governing the allocation of ground-waters, via their transcendence from stream water rights doctrines, is that much of the regulatory structure for ground-water quality followed a similar developmental pattern; that is, many concerns initially voiced for stream quality were later echoed for ground-water quality. For example, the so called Priority Pollutant List of 129 contaminants, which is routinely analyzed for in ground-water contamination investigations, was the result of a legal settlement regarding stream quality parameters (Branter and others, 1981).

1.1.2 Ground-Water Quality Laws

Several governmental units have monitored the quality of fresh waters in the U.S. during the last century, notably the U.S. Geological Survey (USGS), the U.S. Public Health Service (PHS), the U.S. Soil Conservation (SCS), and their State agency counterparts. However, their efforts were limited to analyses for simple inorganic species, bacteria, and physical measurements; identification and control of sources of contamination were generally outside their jurisdiction. Protection of ground water quality by regulation of waste disposal practices has evolved almost entirely within the last fifteen years, concomitant with the creation of the U. S. Environmental Protection Agency (EPA) and the growth in its responsibilities as embodied in public laws (Figure 1.2). Translation of the particulars of these major U.S. Congressional Acts into effective regulations has transpired almost wholly within the last ten years; the following discussion will focus on their respective roles in providing protection of ground-water quality.

Some work on ground-water quality regulations was done by the EPA under the Clean Water Act of 1972 (CWA), particularly under Sections 208 and 303 which concerned area wide planning, surface water quality standards, and the National Pollutant Discharge and Elimination System. As a result of preparing surface water quality standards pertinent to stream wasteload allocations regulated under CWA, EPA published a list of 129 chemicals as 'priority pollutants' to be monitored for in affected receiving waters (Figure 1.3). That list of priority pollutants has been tacitly adopted by EPA for use in permitting and enforcement actions

MAJOR U.S. LAWS AFFECTING GROUND WATER QUALITY

1970 ▪ NATIONAL ENVIRONMENTAL POLICY ACT
NEPA requires environmental impact assessnents for major federally funded projects.

1972 ▪ CLEAN WATER ACT
CWA created permitting system for stream discharges and resulted in list of 129 priority pollutants.

1974 ▪ SAFE DRINKING WATER ACT
SDWA established public water supply standards, Sole Source Aquifer, and Underground Injection Control.

1976 ▪ RESOURCE CONSERVATION AND RECOVERY ACT
RCRA defined hazardous wastes, implemented tracking system, and listed site monitoring and closure criteria.

1980 ▪ COMPREHENSIVE ENVIRONMENTAL RESPONSE, COMPENSATION, AND LIABILITY ACT (SUPERFUND)
SUPERFUND is designed to fund immediate cleanups at abandoned hazardous waste sites with revolving fund.

Figure 1.2. Major U.S. Laws Affording Ground-Water Protection

ACID EXTRACTIBLES

2,4,6-TRICHLOROPHENOL
P-CHLORO-M-CRESOL
2-CHLOROPHENOL
2,4-DICHLOROPHENOL
2,4-DIMETHYLPHENOL
2-NITROPHENOL
4-NITROPHENOL
2,4-DINITROPHENOL
4,6-DINITRO-O-CRESOL
PENTACHLOROPHENOL
PHENOL

VOLATILE ORGANICS

ACROLEIN
ACRYLONITRILE
BENZENE
CARBON TETRACHLORIDE
CHLOROBENZENE
1,2-DICHLOROETHANE
1,1,1-TRICHLOROETHANE
1,1-DICHLOROETHANE
1,1,2-TRICHLOROETHANE
1,1,2,2-TETRACHLOROETHANE
CHLOROETHANE
CHLOROFORM
1,1-DICHLOROETHYLENE
1,2-TRANS-DICHLOROETHYLENE
1,2-DICHLOROPROPANE
CIS-1,3-DICHLOROPROPENE
METHYLENE CHLORIDE
METHYL CHLORIDE
METHYL BROMIDE
BROMOFORM
BROMODICHLOROMETHANE
TRICHLOROFLUOROMETHANE
DICHLORODIFLUOROMETHANE
DIBROMOCHLOROMETHANE
TETRACHLOROETHYLENE
TOLUENE
TRICHLOROETHYLENE
VINYL CHLORIDE
BIS (CHLOROMETHYL) ETHER
2-CHLOROETHYL VINYL ETHER
TRANS-1,3-DICHLOROPROPENE
ETHYLBENZENE

Figure 1.3. EPA List of 129 Priority Pollutants
(a) Acid Extractables and Volatiles

BASE/NEUTRAL EXTRACTIBLES

ACENAPHTHENE
BENZIDINE
1,2,4-TRICHLOROBENZENE
HEXACHLOROBENZENE
BIS(2-CHLOROETHYL) ETHER
2-CHLORONAPHTHALENE
1,2-DICHLOROBENZENE
1,3-DICHLOROBENZENE
1,4-DICHLOROBENZENE
3,3-DICHLOROBENZIDINE
2,4-DINITROTOLUENE
2,6-DINITROTOLUENE
1,2-DIPHENYLHYDRAZINE
FLUORANTHENE
4-CHLOROPHENYL PHENYL ETHER
4-BROMOPHENYL PHENYL ETHER
BIS(2-CHLOROISOPROPYL) ETHER
BIS(2-CHLOROETHOXY)METHANE
HEXACHLOROBUTADIENE
HEXACHLOROCYCLOPENTADIENE
ISOPHORONE
NAPHTHALENE
NITROBENZENE
N-NITROSODIMETHYLAMINE
N-NITROSODI-N-PROPYLAMINE
N-NITROSODIPHENYLAMINE
BIS(2-ETHYLHEXYL) PHTHALATE
N-BUTYL BENZYL PHTHALATE
DI-N-BUTYL PHTHALATE
DI-N-OCTYL PHTHALATE
DIETHYL PHTHALATE
DIMETHYL PHTHALATE
BENZO(A) ANTHRACENE
BENZO(A) PYRENE
BENZO(B) FLUORANTHENE
BENZO(K) FLUORANTHENE
CHRYSENE
ACENAPHTHYLENE
ANTHRACENE
BENZO(GHI) PERYLENE
FLUORENE
PHENANTHRENE
1,2,5,6-DIBENZANTHRACENE
INDENO(1,2,3-CD)PYRENE
PYRENE
TCDD

Figure 1.3. EPA List of 129 Priority Pollutants
(b) Base Neutrals

PESTICIDES

- ALDRIN
- CHLORDANE
- DIELDRIN
- 4,4'DDT
- 4,4'DDE
- 4,4'DDD
- ALPHA ENDOSULFAN
- BETA ENDOSULFAN
- ENDOSULFAN SULFATE
- ENDRIN
- ENDRIN ALDEHYDE
- HEPTACHLOR
- HEPTACHLOR EPOXIDE
- ALPHA BHC
- BETA BHC
- GAMMA BHC (LINDANE)
- DELTA BHC
- TOXAPHENE
- PCB 1016
- PCB 1221
- PCB 1232
- PCB 1242
- PCB 1248
- PCB 1254
- PCB 1260

METALS

- ANTIMONY
- ARSENIC
- BERYLLIUM
- CADMIUM
- CHROMIUM
- COPPER
- LEAD
- MERCURY
- NICKEL
- SELENIUM
- SILVER
- THALLIUM
- ZINC

MISCELLANEOUS

- CYANIDE
- PHENOLICS (AAP)
- ASBESTOS

Figure 1.3. EPA List of 129 Priority Pollutants
(c) Pesticides, Metals, and Miscellaneous

under other environmental statutes that heavily emphasize the protection of ground water from contamination.

The Safe Drinking Water Act of 1974 (SDWA) included the first major legislative mandates to address ground-water quality. Section 1424(e) of that Act provides for the designation of Sole Source Aquifers which may be protected by controlling certain waste disposal activities on or near the recharge zone of aquifers when projects requiring federal government assistance may result in a significant hazard to public health. To qualify for designation as a Sole Source Aquifer the aquifer must be shown to provide the principal source of drinking water; specifically, greater than 50% of the drinking water used by the affected population must be ground water.

The possible mechanisms for designation are many, but all require a substantial show of interest by the affected population. Normally this amounts to petitioning the EPA Regional Administrator by local governments, citizen's groups, or environmental protection advocacy groups. Studies of the aquifer's physical and chemical characteristics, its vulnerability to contamination, and possible remedial and control mechanisms are then conducted under EPA oversight. Much of the detailed work of the studies is performed by local health departments, state geological and water resource agencies, and USGS. To be effective, the regulatory controls instituted by EPA must be echoed by appropriate legislation and ordinances at the State and local levels. Otherwise, developers may seek private/commercial funds that do not carry the constraints imposed on federal funding sources by designation.

The Underground Injection Control program called for by SDWA governs disposal of wastes by injection wells. Five classes of such wells are recognized and regulated according to their operations. Class I injection wells are those that inject hazardous wastes into deep brackish water formations that have adequate confining layers to isolate them from overlying fresh water aquifers. Class II injection wells are those associated with oil and gas production (e.g., brine disposal wells). Class III injection wells are those used to inject in-situ mining solutions and non-hazardous wastes resulting from other energy related industries.

Class IV injection wells are banned, as this category is comprised of hazardous waste injection wells which dispose of wastes directly into fresh water aquifers (broadly defined as those aquifers having less than 10,000 ppm total dissolved solids). This category was created to provide a mechanism for phasing-out such wells as may already be in existence, whereas wells belonging to the preceding three classes are issued permits specifying operational and abandonment conditions.

A fifth category, Class V injection wells, is reserved for those disposal wells which are still under study by EPA, such as common drywells for storm water runoff, irrigation return-flow wells, and septic disposal wells. The contaminants associated with this class are normally found to occur at less than significant levels for health concerns, though these disposal wells may contaminate water supplies beyond tolerable limits if they are present in large numbers (e.g., nitrates from septic disposal wells, salts from stormwater drywells, and pesticides from irrigation return-flow wells). Current studies are focusing on further definition of possible impacts to ground-water quality and on development of appropriate regulatory mechanisms for control of existing problems.

The Resource Conservation and Recovery Act of 1976 (RCRA) was intended to cause a reduction of hazardous waste production by encouraging manufacturing process changes and by conservation. In regulatory terms it consists largely of a waste tracking (manifest) and permit issuance system, which incorporates stringent requirements for the storage, transportation, and disposal of hazardous wastes. Three important features of RCRA are the definition of which wastes are considered hazardous, the requirement that storage, treatment, and disposal facilities have functional ground-water protection and monitoring systems, and the requirement that corrective action plans be developed and implemented for contamination at existing facilities. Operational and closure conditions are specified in legally binding RCRA permits, the violation of which can result in criminal as well as civil penalties. When a waste facility reaches the end of its operational life, monitoring for ground-water impacts is normally required for thirty years; the frequency of monitoring during operation and post-closure depend to a large degree on the nature of the wastes, proximity to water supply wells, and monitoring history.

Significant amendments to RCRA were enacted in 1984. Among these are the establishment of a program to address the problem of leaking underground storage tanks, strict criteria for siting new solid waste landfills, and guidelines for acceptable corrective action plans for contamination at existing facilities. In addition, the hazardous waste disposal wells previously regulated under SDWA (Class-1 UIC wells) were brought under the jurisdiction of RCRA. One of the important aspects of this shift of responsibility is that a significantly greater demonstration of the ability of hazardous waste injection wells to perform with minimal environmental consequences is required. Specifically, this means that use of such disposal well systems will be banned unless it can be proven by March, 1988 that injected wastes cannot escape the zone of emplacement until after they have degraded to harmless residues.

Other changes mandated by the 1984 Amendments strengthened existing provisions of RCRA. For example, generators that produce as little as 220 pounds (100 kilograms) of hazardous waste per month are now subject to RCRA; the original limit was ten times that amount per month. Formerly, waivers from ground-water monitoring requirements were available to the land-disposal units that had double liners and leachate collection systems, and were located above the seasonal high-water table; such waivers are no longer available. All waste facilities must now have double-liners, leachate collection systems, and extensive ground-water monitoring programs. Landfilling bulk or noncontainerized liquids is now banned, as is landfilling of dioxin-containing wastes, spent solvents, wastes containing PCB's at levels greater than 50 parts per million (ppm), highly acidic wastes (pH lower than 2.0), and wastes containing free cyanide at levels greater then 1,000 ppm (U.S. EPA, 1985).

The Comprehensive Environmental Response, Compensation, and Liability Act of 1980 (CERCLA or 'Superfund') was designed to provide immediate clean-up of abandoned hazardous waste sites and those sites that are under the control of parties financially incapable of remedying environmental damage and health risks associated with their facility. The initial mechanism of endowing Superfund with sufficient funds to begin clean-ups at those sites which presently pose an "imminent and substantial endangerment to human health or the environment" was a tax levied on chemical manufacturers and related industries. That mechanism produced a 1.6 billion dollar fund on which the EPA could draw to effect immediate clean-ups while seeking to identify potentially responsible parties (U.S. EPA, 1982). The 'potentially responsible parties' include past and present site owners and operators, and companies that generated and transported the wastes. EPA attempts to recover the funds it expends through out-of-court negotiations or lawsuits with the responsible parties. Punitive assessments equal to three times the actual cost of remediation can be levied for willful neglect by the responsible parties.

EPA has identified approximately 22,000 sites that may be eligible for Superfund clean-ups. To qualify for a Superfund clean-up, a site must be shown to have potentially contaminated the environment to the degree that it may pose a substantial threat to human health and welfare, or the environment. This involves ranking factors such as volume and toxicity of wastes present, condition of waste containers, depth to drinking water, distance from water supply wells, and size of population potentially affected. Those sites that qualify are placed on EPA's National Priority List. By the end of CERCLA's authorized five-year life, approximately 840 sites were on the National Priority List. Long-term cleanups were being conducted at 134 of those sites, with another 317 sites undergoing engineering studies (C&EN staff, 1985a). The cost of properly investigating such sites typically exceeds one million dollars; the cost of complete remediation at any site is often tens of millions of dollars.

In anticipation of re-authorization of CERCLA, there has been continuing disagreements as to the true extent of the problem and the monies that will be required to solve it (C&EN staff, 1985b). The issue is so politically charged that the U.S. Congress' Senate and House of Representatives were unable to resolve major differences in their respective proposals for reauthorization prior to expiration of the original Act. Both proposals called for dramatic increases in funding (6 to 10 billion dollars, as compared with the original 1.6 billion dollars); but major differences in the percentage of the fund to be paid by industry taxes, the limits on liability, the use of innovative technologies in clean-ups, and the appropriations for research existed. As a result, EPA continued programs that depended on CERCLA reauthorization at a much reduced rate during late 1985 through early 1986, using monies that had been carried over from previous fiscal years. When those funds ran out in March, 1986, Congress authorized a limited (60 day) extension of CERCLA with supplemental funds. This stop-gap measure did not buy sufficient time for Congress to resolve the issue, however. As of this writing, additional stop-gap measures are being used to fund Superfund activities; it is expected that this will be the case until after the November, 1986 Congressional elections.

Other important laws which afford elements of ground-water protection are the Surface Mining Control Act (SMCRA), the Federal Insecticide, Rodenticide, and Fungicide Act (FIFRA), and the Toxic Substances Control Act (TSCA). SMCRA requires preventative measures to be taken during mining and requires return of locally affected lands and water to as near pre-mining conditions as possible after cessation of operations. FIFRA and TSCA require pre-manufacturing tests for contaminant persistence and mobility in the environment; intended use and application levels are regulated.

1.1.3 The Ground-Water Protection Strategy

Emphasis has recently been given to coordination of the various EPA programs dealing with ground water. Toward this goal EPA's Office of Ground-Water Protection published the Ground-Water Protection Strategy. The key components of the Strategy are (U.S. EPA, 1984):

1. EPA will provide support to States for short-term build-up of programs and institutions at the State level. The funds are to be made available through existing grant programs and must be used to provide development and planning of ground-water protection programs. Specifically, the creation of needed data systems, the assessment of legal and institutional impediments, and the development of permitting systems and aquifer classification systems are to be encouraged.
2. EPA will seek to address those potential sources of contaminants that are not currently regulated. In particular, detailed assessments are to be conducted on the problem of leaking underground storage tanks, and existing programs to examine the impacts of surface impoundments and landfills will be expanded.
3. EPA will issue guidelines for its decisions affecting ground-water protection and cleanup efforts. The guidelines

will be based on a policy which requires that ground-water protection and cleanup decisions consider the highest beneficial use to which ground water can presently or potentially be put. A classification system is being recommended by EPA so that an appropriate level of protection can be extended to each aquifer (Figure 1.4).

4. EPA will strengthen its internal organization to better coordinate its various ground-water protection programs. The first steps in this direction were the establishment of the Office of Ground-Water Protection on April 2, 1984, and its subsequent publication of the Ground-Water Protection Strategy. Further efforts in this direction involve the creation of regional liasons and a special Ground-Water Steering Committee to review all ground-water policies and regulations. Moreover, an Interagency Committee on Ground Water has been created to coordinate EPA efforts with those of the USGS, DOE, NRC and other agencies.

EPA has been and will continue to conduct advanced research studies on the sources, transport mechanisms, and fate of contaminants. This is the prime mechanism by which regulatory decisions are expected to improve. Other federal governmental units have complimentary programs that address ground-water issues. The U.S. Geological Survey (USGS) develops basic information on water supply potentials of aquifers and on fundamental geologic and hydrologic processes which are germane to ground water issues.

EPA GROUND-WATER STRATEGY
RECOMMENDED AQUIFER CLASSIFICATIONS

CLASS I

MUST BE HIGHLY VULNERABLE TO CONTAMINATION, HYDROGEOLOGICALLY
(E.G., SAND AND GRAVEL AQUIFERS AND KARST AQUIFERS)

AND, EITHER

(1) AN IRREPLACEABLE SOURCE OF DRINKING WATER,
(SIMILAR TO SOLE SOURCE AQUIFERS)

OR

(2) ECOLOGICALLY VITAL (E.G., PROVIDES BASEFLOW TO
SENSITIVE STREAMS, OR SUPPORTS WETLANDS HABITATS)

SPECIAL RESTRICTIONS ON WASTE STORAGE AND DISPOSAL WILL
BE INVOKED UNDER RCRA, CERCLA, UIC, TSCA, etc.

CONTAMINATION CLEAN-UPS TO BE MAXIMUM POSSIBLE

CLASS II

INCLUDES ALL GROUND WATER CURRENTLY OR POTENTIALLY USEFUL
AS A DRINKING WATER SUPPLY, AND NOT COVERED BY CLASS I

ADDITIONAL SITING RESTRICTIONS FOR NEW RCRA FACILITIES;
UIC, CERCLA, TSCA, etc. TO OPERATE PER EXISTING REGULATIONS

CONTAMINATION CLEAN-UPS TO BACKGROUND LEVELS, DRINKING WATER
STANDARDS, OR ALTERNATE CONCENTRATION LEVELS (ACL's)

CLASS III

INCLUDES GROUND WATERS THAT HAVE TDS >10,000 PPM, AND
THOSE THAT ARE HIGHLY CONTAMINATED BY NATURAL OR NONPOINT SOURCES

MINIMAL REGULATIONS AND MINIMAL CLEAN-UPS

Figure 1.4. U.S. EPA's Recommended Aquifer Classification Scheme

Where energy-related wastes are concerned, the U.S. Department of Energy (DOE) has a commitment to develop information on potential impacts to ground water. The U.S. Congress has directed DOE to provide a balance between the need for adequate energy development and use, and environmental protection. Energy development and use issues are handled almost exclusively by DOE, whereas EPA is heavily involved in cooperative ventures with DOE regarding potential environmental impact assessments. For example, a cooperative study with EPA examined the potential impacts to ground water quality by coal strip mining (Hounslow and others, 1979).

In similar efforts, DOE maintains a close working relationship with the U.S. Nuclear Regulatory Commission (NRC), particularly with regard to the mandates of the 1982 Nuclear Waste Policy Act. The Act calls for the siting and licensing of two geologic repositories by DOE for disposal of high level radioactive waste beginning in 1998. Slow development of safe repositories for high-level radioactive waste has limited the evolution of an associated permitting system for final disposal operations (OTA, 1985). NCR and DOE are conducting transport and fate research of radionuclide migration relative to such disposal operations in support of requisite risk analyses associated with nuclear repositories. Establishment of suitable health and environmental criteria for disposal of nuclear wastes is being carried out cooperatively by all the involved agencies.

Numerous non-governmental entities conduct useful programs devoted to ground-water quality, as well. Some, such as the Electric Power Research Institute (Palo Alto, California), have research budgets rivalling those of the largest public agencies. Other private institutes and nonprofit organizations (e.g., universities, endowments, and scientific societies and associations) produce considerable ground-water quality research, and some consulting firms also engage in such work routinely. Hence, the scientific base on which protection of ground-water quality depends continues to expand in all areas of both the private and public sectors.

1.2 Purpose of This Work

Reflecting on the rapid growth of laws and regulations that address ground-water quality, it is evident that it will continue to be a major societal concern for many years to come. Prevention and remediation of ground-water contamination and its effects will occupy the time of an increasing number of ground-water scientists and engineers, health specialists, and policy-makers. This work is designed to provide helpful suggestions for the development of solutions to problems those individuals will face. These suggestions principally relate to the need for high quality data, an understanding of the limitations of existing theories of ground-water and contaminant behavior, and the interplay between technical and sociopolitical factors in the decision-making process.

It is hoped that this work will encourage technical specialists and policy-makers alike to develop more sensitivity to the difficulties involved in performing ground-water contamination assessments. Those difficulties are as much political as they are technical, and to overcome them requires skill in both arenas. That there are no clear, all-encompasing technical solutions to most ground-water contamination problems is rarely recognized by policy-makers. That technical issues often must be wholly subordinate to political expediencies is just as poorly appreciated by technical specialists. These are matters that are rarely taught outside the school of life, but that bear some discussion nevertheless. Consequently, a conscious attempt has been made to place the technical discussions provided here within the broader framework of the decision-making process.

In that context, a special effort is made here to debunk several misconceptions regarding current abilities to address ground-water contamination problems. Predominant among these misconceptions are:

1. The appropriate tools exist for collection of the data, coefficients, and parameters needed in ground-water contamination assessments; for qualitative estimates, at least.
2. Research efforts need focus only on improving the quantitative accuracy of assessments; existing theories adequately represent the natural processes that affect contaminant transport.
3. The conventional approach to characterization of ground-water problems utilizes the best available technologies.
4. A ground-water contamination problem can be sufficiently characterized by screening-level studies to enable the selection of the appropriate remedial actions; more detailed investigations are often unnecessary, and can be delayed until the design phase.
5. Detailed investigations are not cost-effective; advanced field tests are simply too expensive.

6. Remedial action plans typically have a high probability of success; very few will perform unacceptably, because specialists designed them.
7. Traditional relationships and communication within and between groups of specialists of various disciplines, management, attorneys, and legislators will suffice to address this problem, just as occurred with other environmental media.

1.3 Scope of Topics Addressed

A broad introduction to the need for ground-water contamination assessments has been given, by way of descriptions of the laws and regulatory policies devoted to prevention and remediation of ground-water contamination. To address the mechanics of ground-water contamination assessments, it is necessary to examine the science on which they are based. The primary thrust of this document does not require a lengthy review of the derivations of the applicable scientific laws and empirical relationships. Rather, an examination is made of those elements of scientific concepts that are still evolving, to underscore the magnitude of errors associated with the assumptions and simplifications invoked by existing theories.

With that foundation, the methods by which the necessary data, coefficients, and parameters are collected and interpreted are discussed. Those discussions focus on the quality of the data for specific uses; noting that the use dictates the relative value, not the mathematical sophistication or intrinsic features of the data. This begins with an examination of strategies for selecting monitoring well locations and sampling frequencies, drilling methods, and sampling techniques. These discussions are followed by an explanation of some of the field tests that are useful for estimating aquifer parameters. A few mathematical models are then introduced to serve as vehicles for discussions of the sensitivity of interpretations to estimates of key parameters.

Case histories are presented in a subsequent chapter to further underscore the need to conceptualize each problem correctly, to illustrate commonly made mistakes, and to describe the interdependency of technical and non-technical factors in decision-making. The case histories are sufficiently varied and complex to show the need for improved theoretical understandings of the natural processes that affect contaminant transport in the subsurface. They also serve to emphasize the desireability of more detailed field characterization efforts.

Finally, a summary is provided that condenses much of the preceeding material, and goes on to phrase the conclusions in terms of implications for decisionmakers. Recommendations for research are cited separately, with annotations as to the expected utility of the desired products. Sources of additional information and listings of the computer codes for the mathematical models introduced in the text (with additional simulations) are appendicised.

CHAPTER II

OVERVIEW OF CONTAMINANT TRANSPORT PROCESSES

2.1 Historical Development and Current Approaches

Much of the attention devoted to natural processes in the subsurface has come in the form of applying concepts derived from work in other environmental media. Theories of organic chemical sorption to stream sediments, for example, have been applied to subsurface contaminant transport problems without serious modification (Karickhoff and others, 1979; Schwarzenbach and Westall, 1981; Karickhoff, 1984). The solution to the hydrodynamic flow equation for drawdown around a well that penetrates an aquifer of effectively infinite areal extent (no natural boundary effects) was derived by analogy to heat flow theory for a cylindrical heat sink in a uniform slab (Theis, 1935). The theory of the compressibility and elasticity of artesian aquifers was derived, in part, by invoking the principles of consolidation developed in soil mechanics (Meinzer, 1928).

Many of the natural processes that affect contaminant transport in the subsurface, however, have no direct analogs in other environmental media. Dispersion in ground water differs markedly from dispersion in surface waters, for example, because the turbulent mixing that is so effective in surface waters is absent in ground waters (karst aquifers are the notable exception). Pipe-flow theory has not been found adequate to represent the flow through fractured rocks because it does not consider the interdependencies of flow in the fractures and flow in the bulk matrix of the rock. Hence, substantial modification of borrowed theories and invention of new theories are required for proper representation of natural processes that affect subsurface contaminant transport.

2.1.1 Levels of Understanding

Only recently have substantial advances been made in understanding subsurface transport processes. As a consequence, state-of-the-art in ground-water contamination studies lags behind 'state-of-the-science'. That is, what is actually put into routine use differs from accepted theory because of simplifications, difficulties in measuring certain coefficients and constants, and so on. This is illustrated, for instance, by the use of linearized sorption constants for contaminants shown to produce nonlinear isotherms. Similarly, state-of-the-science in ground-water contamination studies lags behind 'state-of-the-knowledge', or what has been shown to occur despite the lack of theories that can adequately describe or predict it. For instance, field observations indicate that sorbed contaminants may be transported through the subsurface in association with ultra-fine particles. If this is indeed occurring, it would mean that movement of the particles onto which highly retarded organic chemicals are sorbed could be more effective in transporting those contaminants than the dissolved species movement itself.

It is therefore useful to view subsurface contaminant transport on these three levels of understanding: conceptual (state-of-the-knowledge), theoretical (state-of-the-science), and practical (state-of-the-art). This discussion will focus on changing perceptions of how natural processes control the transport and fate of contaminants in the subsurface. Specifically, emphasis will be placed on the need for fresh conceptualizations of transport mechanisms in the subsurface to overcome limitations imposed by current theoretical assumptions. How simplifications of some natural process theories complicate others in practical applications will also be highlighted. Finally, the usefulness of tentative research results in developing better understandings of cause-and-effect relationships for site-specific problems will be noted.

2.1.2 Conceptualizations of Transport Processes

There are many natural processes that affect chemical transport from point to point in the subsurface. These natural processes can be arbitrarily divided into three categories: physical, chemical, and biological (Table 2.1). Conceptually, contaminant transport in the subsurface is an undivided phenomenon composed of these processes and their interactions. Theoretically, only certain processes can be represented currently, and their interactions are usually ignored. For example, the presumed additive effects that increasingly complex natural processes would have on the concentration distribution of common solvents (e.g., trichloroethene) downgradient from continuous and slug-release sources is illustrated in Figure 2.1.

NATURAL PROCESSES AFFECTING SUBSURFACE CONTAMINANT TRANSPORT

PHYSICAL PROCESSES

- **Advection (porous media velocity)**
- **Hydrodynamic Dispersion**
- **Molecular Diffusion**
- **Density Stratification**
- **Immiscible Phase Flow**
- **Fractured Media Flow**

CHEMICAL PROCESSES

- **Oxidation—Reduction Reactions**
- **Radionuclide Decay**
- **Ion—Exchange**
- **Complexation**
- **Co—Solvation**
- **Immiscible Phase Partitioning**
- **Sorption**

BIOLOGICAL PROCESSES

- **Microbial Population Dynamics**
- **Substrate Utilization**
- **Biotransformation**
- **Adaptation**
- **Co—metabolism**

Table 2.1. Natural Processes That Affect Contaminant Transport

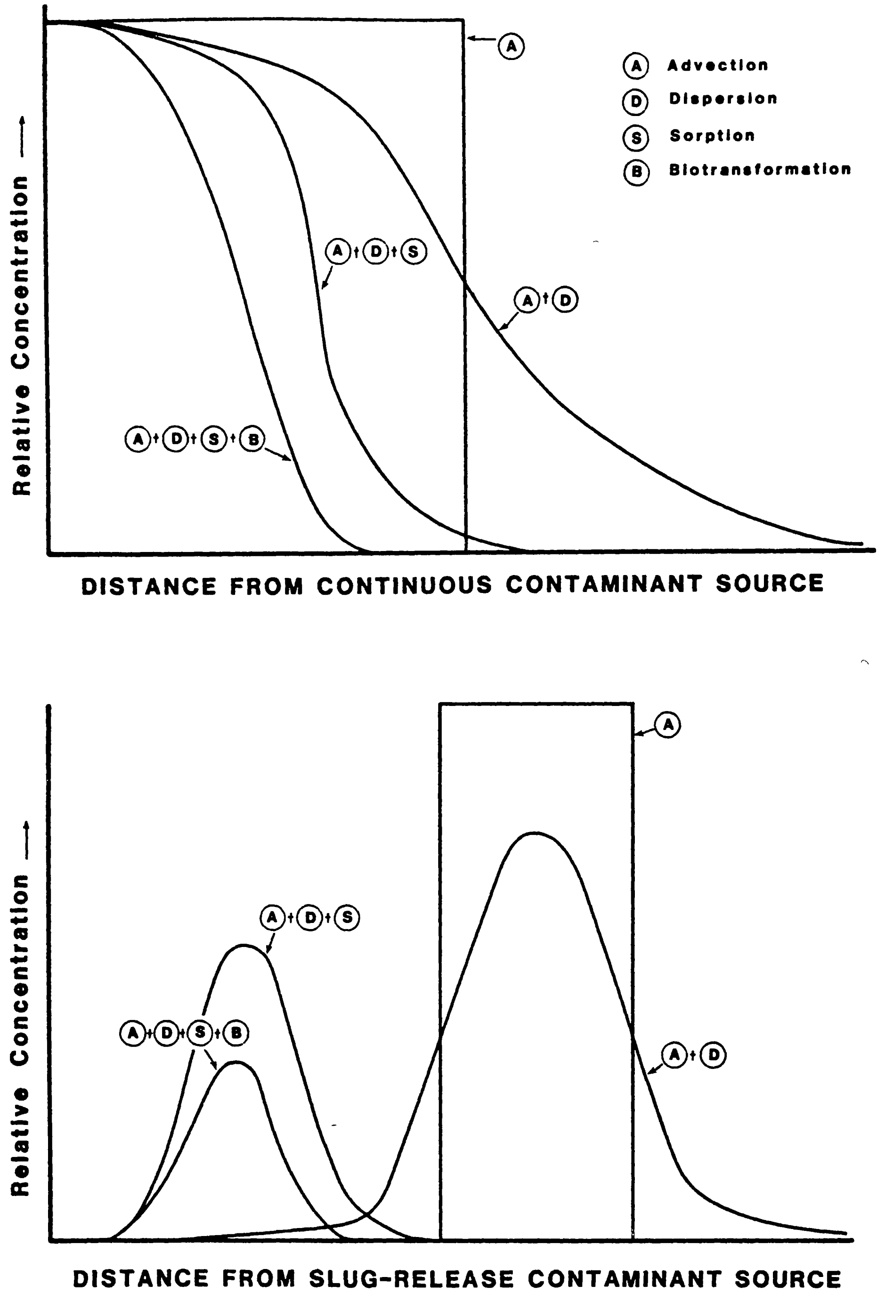


Figure 2.1. Effects of Natural Processes on Contaminants Released from Continuous and Slug-Release Sources

Estimation of a concentration distribution by velocity considerations (advection) alone would predict a sharp concentration front, with concentrations throughout the plume equal to the input concentration (assuming uniform flow in a homogenous, isotropic stratum). Incorporation of dispersive effects would effectively dilute the plume by spreading the mass out according to a gaussian distribution of velocities (where the average velocity equals that for the advective case). The concentration front would necessarily extend farther downgradient than by the advective estimate; however, the total mass of contaminant in solution would be identical by either estimation method. If the effects of sorption were also considered, mass would be removed from solution so the area under the curve shown in Figure 2.1 is less for this case than for the previous estimation methods. Total mass of the specific contaminant in the system would be identical, however, because the mass removed would remain intact while on the aquifer solids.

Sorption would also serve to retard the movement of the plume. Depending on the degree of sorption, the concentration front may arrive ahead of or behind the point predicted by advection alone; but it would always fall short of the point predicted by the combined advection-dispersion estimate. Biotransformation, radiodecay, and certain complexation and precipitation reactions would remove mass from solution too, but the removal would not be reversible. The total contaminant mass in the system would not be constant, but would decline. Hence, the concentration front of the specific contaminant would eventually stop advancing (total mass depletion).

In real-world settings, the overall transport process is not merely the sum of the effects of the individual natural processes, but is gestalt. The sum of its parts do not equal the whole because of interactions between the parts. Appreciation of this fact is easy conceptually, as empirical observations afford evidence that interactions between the natural processes can be significant. It is known, for example, that microbial populations indigenous to the subsurface are found almost exclusively on the aquifer solids (Wilson and others, 1983); this implies that sorption of contaminants to those solid surfaces may be necessary prior to their degradation by the microbiota (analagous to biofilm reactors). It is also known that mechanical diffusion of oxygen often limits the rate of biotransformations (Bedient and others, 1984). Quantification of such interactions and translation of these conceptualizations into workable theory, however, are stumbling blocks. Numerous experiments will be necessary to elucidate the detailed mechanisms of such interactions, and few of these are currently underway.

2.1.3 Practical Applications

In the present theoretical context, a collection of scientific laws and empirically derived relationships comprise representations of the overall transport process. As expected, the universally shared premise that underlies most existing theoretical expressions is that there are no interactions between various natural processes, measureable or otherwise. Significant errors may thus result from the discrepancy between current conceptual and theoretical approaches.

Practical applications of theoretical expressions often require that further assumptions and simplifications be made in order to obtain mathematically tractable solutions. These assumptions and simplifications can cause substantial errors, by predicting contaminant behavior that is not faithful to the real-world situation. Because of this, the magnitude of errors that arise from each assumption and simplification must be carefully evaluated. The phrase 'magnitude of errors' deserves emphasis because highly accurate evaluations usually are not possible. Even rough approximations of the errors involved are rarely trivial exercises because they frequently demand estimates of some things that are as yet ill-defined (e.g., dispersivity).

2.2 Physical processes

Until recently, ground-water scientists studied physical processes to a greater degree than chemical or biological processes. This bias resulted in large measure from the fact that, in the past, ground-water practitioners dealt mostly with questions of adequate water supplies. As quality considerations began to dominate ground-water issues, the need for studies of the chemical and biological factors became apparent -- as did the need for more detailed representations of the physical factors.

There are two complimentary ways to view the physical processes involved in subsurface contaminant transport: the piezometric (pressure) viewpoint and the hydrodynamic viewpoint. Ground-water problems of yesterday could be addressed by the former, such as solving for the change in pressure head caused by pumping wells. Contamination problems of today also require detailed analyses of wellfield operations, for example, pump-and-treat plume removals; however, solutions depend principally on hydrodynamic evaluations, such as computing ground-water velocity (advection) distributions and dispersion estimates for migrating plumes.

2.2.1 Advection

General expressions for flow velocities in non-fractured ground-water systems can be obtained by rearrangement of Darcy's Law, as follows (Todd, 1980):

$$Q = K * A * I \qquad \text{[Darcy's Law]}$$

$$Q / A = K * I = V_D \qquad \text{[Darcy 'flux']}$$

$$V = V_D / \emptyset_e \qquad \text{[seepage velocity]}$$

where:
Q is the flowrate (cu.ft./day or cu.m./day),
K is the saturated hydraulic conductivity (ft/day or m/day),
A is the cross-sectional area normal to flow (sq.ft. or sq.m.),
I is the hydraulic gradient (dimensionless),
V_D is the Darcy 'flux' (ft/day or m/day),
$\emptyset_e$ is the effective porosity (dimensionless), and
V is the average flow velocity (ft/day or m/day).

Ground-water velocity distributions can be approximated if the variations in hydraulic conductivity, porosity, and the strength and location of recharge and discharge can be estimated. While there are several field and laboratory methods for estimating hydraulic conductivity, these are not directly comparable because different volumes of aquifer material are affected by different tests. Laboratory permeameter tests, for example, obtain measurements from small core samples and thus give point value estimates. These tests are generally reliable for consolidated rock samples, such as sandstone, but can be highly unreliable for unconsolidated samples, such as sands, gravels, and clays. Pumping tests give estimates of hydraulic conductivity that are averages over the entire volume of aquifer that is subject to the pressure changes induced by pumping. These give repeatable results, but they are often difficult to interpret. Tracer tests are also used to estimate hydraulic conductivity in the field, but are difficult to conduct properly.

Regardless of the estimation technique used, the best that can be expected are order-of-magnitude estimates for hydraulic conductivity at the field scale appropriate for site-specific work. Conversely, porosity estimates that are accurate to better than a factor of two can be obtained. Estimation of the strength of nonpoint sources of recharge to an aquifer, such as infiltrating rainfall and leakage from other aquifers, is another order-of-magnitude effort. Similarly, nonpoint sources of discharge, such as losses to gaining streams, are difficult to quantify. Estimation of the strength of point sources of recharge or discharge, such as injection or pumping wells, can be highly accurate.

Consequently, it is not possible to generalize the quality of velocity distributions; they may be accurate to within a factor of two for very simple aquifers, but are more often accurate to an order-of-magnitude only. This situation has changed little over the past 20 years because better field and laboratory methods for characterizing velocity distributions have not been developed. This, however, is not the primary difficulty associated with defining advective influences in actual ground-water contamination assessments.

The primary difficulty is that field tests for characterizing the physical parameters that control velocity distributions are not incorporated into contamination investigations on a routine basis. The causes seem to be a perception that field tests are too expensive, a perception that mathematical models can 'back-out' an approximation of the velocity distribution (presumably eliminating the need for field tests), and a scarcity of practitioners that are familiar with the appropriate field tests. Because of this situation the conventional approach may not be cost-effective with regard to the remedial action phase or the overall project even though it is cost-effective with regard to the site investigation phase of a project.

A more field-oriented approach to ground-water contamination assessments is preferable because this leads to more certain decisions regarding the design and optimization of remedies. In the absence of field-derived estimates of key parameters, sophisticated mathematical models may be used to provide estimates; but the non-uniqueness of such modeling results has been amply demonstrated (Konikow, 1986). Several combinations of parameter estimates will provide a reasonable fit to observed data, but only one combination corresponds to the real-world case. The predictive accuracy for places or time periods where there are little or no data, then, is usually very poor.

2.2.2 Dispersion

Hydrodynamic dispersion comprises both mechanical mixing and molecular diffusion, and is represented by expressions that are analagous to Fick's first law of diffusion (Freeze and Cherry, 1979; Anderson, 1979):

$$D_x = a * V_x + D^\circ$$

$$D_y = b * V_y + D^\circ$$

$$D_z = c * V_z + D^\circ$$

where:

D_x is the x-axis coefficient of dispersion (sq./day or sq.m./day),

D_y is the y-axis coefficient of dispersion (sq.ft./day or sq.m./day),

D_z is the z-axis coefficient of dispersion (sq.ft./day or sq.m./day),

a is the x-axis dispersivity (ft or m),

b is the y-axis dispersivity (ft or m),

c is the z-axis dispersivity (ft or m),

V_x is the x-axis average velocity (ft/day or m/day),

V_y is the y-axis average velocity (ft/day or m/day),

V_z is the z-axis average velocity (ft/day or m/day), and

D° is the molecular diffusion coefficient (sq.ft./day or sq.m./day).

In essence, the first term of the coefficient in any direction is the product of a characteristic mixing length (the dispersivity) and the average ground-water flow velocity; the second term accounts for molecular diffusion. In practical applications, the first term may be considered negligible for flow systems having extremely slow natural velocities; such as transport through a dense clay stratum, and transport within the bulk matrix of fractured rock aquifers. The second term may be considered negligible when an appreciable natural flow velocity is present, as occurs in most productive aquifers. Dispersion estimates are, therefore, predicated on velocity distribution estimates. Hence, their accuracy is directly dependent on the accuracy of the estimated hydraulic conductivity distribution.

Tracer tests have been the primary method used to determine dispersion coefficients until recently. Presently there are suggestions that any field method capable of generating a detailed understanding of the spatial variability of hydraulic conductivity (which in turn could give an accurate representation of the velocity distribution) may be used to derive estimates of dispersion coefficients. The manner in which data from field tests should be used to derive estimates of dispersion coefficients, however, is a controversial issue. There are both deterministic (Molz and others, 1983) and stochastic schools of thought (Gelhar and Axness, 1983), and neither has been conclusively demonstrated in complex hydrogeological settings.

The effects of advection and dispersion are considered jointly in most models of non-reactive contaminant transport, according to the following expression (Freeze and Cherry, 1979; Mercer and Faust, 1981):

$$[(D_x * (\delta^2C/\delta x^2)) + (D_y * (\delta^2C/\delta y^2)) + (D_z * (\delta^2C/\delta z^2))]$$
$$- [(V_x * (\delta C/\delta x)) + (V_y * (\delta C/\delta y)) + (V_z * (\delta C/\delta z))]$$
$$= \delta C/\delta t$$

where: C is the fluid phase concentration (lbs/ft^3 or µg/L),

and

t is time (days).

In this 'advection-dispersion' equation, the first group of terms represents the combined effects of dispersion and the spatial rate of change of contaminant concentrations. When the combined effects of advection and the nominal concentration gradient (the second group of terms) are subtracted, the time rate of change of contaminant concentration is obtained for any point of interest. Judging from the predominance of the velocity terms, and the inherent role of velocity in the dispersive terms, the key to successful application of this equation would seem to be proper definition of the velocity distribution.

2.2.3 Complicating Factors

Certain subtleties of the spatial variability of hydraulic conductivity must be understood because of its key role in the determination of velocity distributions and dispersion coefficients. Hydraulic conductivity is also known as the coefficient of permeability because it is comprised of fluid factors as well as the intrinsic permeability of the stratum in question (Todd, 1980; Freeze and Cherry, 1979):

$$K = k * (\rho g/\mu)$$

where: K is the hydraulic conductivity ('coefficeint' of permeability),

k is the intrinsic permeability

p is the fluid density,

g is the acceleration of gravity, and

μ is the fluid viscosity.

This means that a stratum of uniform intrinsic permeability (which depends strictly on the arrangement of its pores) may have a wide range of hydraulic conductivity because of differences in the density and viscosity of fluids that are present. The result is a dramatic downward shift in local flow directions near plumes that have as little as a 1% increase in density relative to uncontaminated water. Such density contrasts frequently occur at landfills and waste impoundments. It is often necessary to correct misimpressions of the direction of a plume's movement because density considerations were not addressed.

Many solvents and oils are highly insoluble in water, and may be released to the subsurface in amounts sufficient to form a separate fluid phase. Because that fluid phase will probably have viscosity and density different from freshwater, it will flow at a rate and, possibly, in a direction different from that of the freshwater with which it is in contact. If an immiscible phase has a density approximately the same or less than that of ground water, this phase will not move down past the capillary fringe of the ground water. Instead, it will flow along the top of the capillary fringe in the direction of the maximum water-level elevation drop.

If the density of an immiscible phase is substantially greater than the ground water, the immiscible phase will push its way into the ground water as a relatively coherent blob. The primary direction of its flow will then be down the dip of the first impermeable stratum encountered. There is a great need for better means of characterizing such behavior for site-specific applications. Currently, estimation methods are patterned after multiphase oil reservoir simulators (Hochmuth and Sunada, 1985; Abriola and Pinder, 1984). One of the key extensions needed is the ability to predict the transfer of trace levels of contaminants from immiscible fluids to ground water, such as aromatic components from gasoline (Baehr and Corapcioglu, 1984).

Anisotropy is a subtlety of hydraulic conductivity that relates to structural trends of the rock or sediments of which an aquifer is composed. Permeability and hydraulic conductivity are directionally dependent in anisotropic strata, e.g.:

$$Q_x = K_x * A_{yz} * (dh/dx) = k_x(pg/\mu) * A_{yz} * (dh/dx)$$

$$Q_y = K_y * A_{xz} * (dh/dy) = k_y(pg/\mu) * A_{xz} * (dh/dy)$$

$$Q_z = K_z * A_{xy} * (dh/dz) = k_z(pg/\mu) * A_{xy} * (dh/dz)$$

When molten material from deep underground crystallizes to form granitic or basaltic rocks, for instance, it forms cleavage planes which may later become the preferred directions of permeability. Marine sediments accumulate to form sandstone, limestone, and shale sequences that have much less vertical than horizontal permeability. The seasonal differences in sediments that accumulate on lakebeds, and the stratification of grain sizes deposited by streams as they mature, give rise to similar vertical-to-horizontal anisotropy. Streams also cause anisotropy within the horizontal plane, by forming and reworking their sediments along a principal axis of movement. These structural variations in permeability would be of minimal concern except that water does not flow at right angles to water-level elevation contours under anisotropic conditions. Instead, flow proceeds along oblique angles, with the degree of deviation from a right-angle pathway proportional to the amount of anisotropy (Fetter, 1981). This fact is all too often ignored and the causes again seem to be a reluctance to conduct the proper field tests, combined with an over-reliance on mathematical modeling.

If the pathways created by cleavage planes and fractures begin to dominate fluid flow through a subsurface stratum, the directions and rates of flow are no longer predictable by the equations used for porous rock and sediments. There have been a number of attempts to represent fractured flow as an equivalent porous medium, but these tend to give poor predictions when major fractures are present (Sudicky and Frind, 1982), when there are too few fractures to guarantee a minimum degree of interconnectedness (Tang and others, 1981), and when diffusion from the bulk matrix is important in controlling contaminant release (Grisak and Pickens, 1980; Neretnieks, 1980; Feenstra and others, 1984; Pankow and others, 1986).

Other representations of fractured media transport that have been studied are various dual porosity models, in which the bulk matrix of the rock has one porosity and the fracture system has another. Further development of the dual porosity approach is limited by the difficulty in determining a transfer function to relate the two different porosity schemes. Research in this area needs to be accelerated because there is a great likelihood of fractured flow in just those situations commonly believed to be the most suitable for disposal of hazardous wastes, such as building landfills on 'impermeable' bedrock.

2.2.4 Considerations for Predictive Modeling

Equations for the combined advection-dispersion process are used to estimate the time during which a nonreactive contaminant will travel a specific distance, the pathway it will travel, and its concentration at any point. The accuracy of most predictions is only fair for typical applications, because of the complexity of the problems and the scarcity of site-specific hydrogeologic data. The lack of such data can be improved on with much less effort and expense than is commonly presumed, especially when costs of another round of chemical sampling are compared with the costs of additional borings, core retrievals, geophysical logging, or permeability testing.

Equations that assume a nonreactive contaminant have limited usefulness, because most contaminants react with other chemical constituents in subsurface waters and with subsurface solids in a manner that affects the rate at which they travel. Nevertheless, nonreactive advection-dispersion equations are often used to generate 'worst-case' scenarios, on the presumption that the maximum transport velocity is obtained (equal to that of pure water). This may not be as useful as it first seems. Remedial action designs cannot be optimized without detailed breakdowns of which contaminants will arrive at extraction wells and when, how long contaminants will continue their slow release from subsurface solids, and whether the contaminants will be transformed into other chemical species by chemical or biological forces. To address these points, special terms must be added to the advection-dispersion equations.

2.3 Chemical Processes

As difficult as the foregoing complications may be, predicting how chemical contaminants move through the subsurface is a relatively trivial matter when the contaminants behave as ideal, nonreactive substances. Unfortunately, such behavior is limited to a small group of chemicals. The actual situation is that most contaminants will, in a variety of ways, interact with their environment through biological or chemical processes. This section focuses on the dominant chemical processes that may ultimately affect the transport behavior of a contaminant.

As with the physical processes previously discussed, some of the knowledge of chemical processes has been translated into practical use in predictive models. However, the science has, in many instances, advanced well beyond what is commonly practiced. Furthermore, there is considerable evidence that suggests that numerous undefined processes affect chemical mobility. Most of the deviation from ideal nonreactive behavior of contaminants relates to their ability to change physical form by energetic interactions with other matter. The physical-chemical interactions may be grouped into alterations in the configuration of the orbital electrons of an element (whether by itself, or as part of a large molecule), alterations in its nuclear composition, the establishment of new associations with other chemical species, and interactions with solid surfaces.

2.3.1 Orbital Electron Alterations

The first of these possible changes is typified by oxidation-reduction or redox reactions. This class of reactions is especially important for inorganic compounds and metallic elements because their reactions often result in changes in solubility, complexing capacity, or sorptive behavior, and these directly impact on the mobility of the chemical. For example, the following expressions describe the generation of acidic waters that typically accompanies the exposure of sulfide minerals to ground water, a problem in the mining of coal and many metallic ores (Manahan, 1975; Stumm and Morgan, 1981; Keely, 1979):

$$2FeS_{2(pyrite)} + 2H_2O + 7O_2 = 4H^+ + 4SO_4^{2-} + 2Fe^{2+}$$

$$4Fe^{2+} + O_2 + 4H^+ = 4Fe^{3+} + 2H_2O$$

$$Fe^{3+} + 3H_2O = Fe(OH)_{3(yellowboy)} + 3H^+$$

$$FeS_{2(pyrite)} + 14Fe^{3+} + 8H_2O = 15\ Fe^{2+} + 2SO_4^{2-} + 16H^+$$

In the first step, pyrite (iron sulfide, or 'fools gold') is oxidized by water and oxygen to yield sulfuric acid and ferrous iron. The second step, the oxidation of ferrous to ferric iron, is the rate-limiting step; being about 100 times slower than the first step despite catalysis by several bacteria (e.g., Metallogenium, Thiobacillus thiooxidans, Thiobacillus ferrooxidans, and Ferrobacillus ferrooxidans). The third and fourth steps are quite rapid, and represent concurrent alternative pathways. Both pathways result in the release of more acid, fueling the cycle. Precipitation of yellowboy (a semigelatinous, amorphous hydroxide of iron) predominates near neutral pH, whereas the further dissolution of pyrite is favored at highly acidic pH. Side effects, such as the leaching of toxic metals from natural and processed ores (mine tailings) by acidic waters, are environmentally significant. The Coeur d'Alene Mining District in northern Idaho, for example, now contains a Superfund site where the mobilities of lead, cadmium, arsenic, and other toxic metals generated by natural leaching of sulfide mineral wastes are the major concern.

Redox reactions are reasonably well understood for simple minerals, but not for synthetic organic chemicals. For example, the interconversion of sulfones, sulfoxides, and sulfides is described as a redox process that needs considerably more study to explain the degradation of certain common pesticides (Macalady and others, 1986). It has been shown to affect the persistence of aldicarb / Temik™ (Ou and others, 1985; Miles and Delfino, 1985) and fenamiphos / Nemacur™ (Lee and others, 1986). Aldicarb has been extensively used to control pests in orchards, and fenamiphos is a proposed substitute for the banned pesticides DBCP (dibromochloropropane) and EDB (ethylenedibromide). There are major practical obstacles to applying these findings; it is difficult to determine the redox state of the aquifer zone of interest and to identify and quantify redox-active reactants involved in the degradation of these compounds (Macalady and others, 1986).

Hydrolysis, elimination, and substitution reactions that affect certain contaminants also belong to the category of orbital electron alterations. The chemistry of many organic contaminants has been well defined in surface water environments (Tinsley, 1979; Stumm and Morgan, 1981; Karickhoff, 1984); however, there are unique aspects to such reactions in the subsurface, not the least of which is long residence time. These direct influences on contaminant mobility deserve increased research efforts. There is also a need to investigate the feasibility of promoting in-situ abiotic transformations that may enhance the potential for biological mineralization of pollutants.

2.3.2 Nuclear Alterations

The second kind of chemical interaction introduced in this section is well understood -- the internal rearrangement of the nuclear structure of an element. Radiodecay occurs by a variety of routes, but the rate at which it occurs is always directly proportional to the number of radionuclide atoms present (Chase and Rabinowitz, 1967). Mathematically, this is stated as a first-order rate law:

$$-dN/dt = kN$$

where: N is the number of atoms of the radionuclide, and

k is the decay constant for that radionuclide.

This fact seems to make mathematical representation in contaminant transport models quite straightforward because it allows characterization of the process with a unique, well-defined decay constant for each radionuclide.

A mistake that may be made when decay constants are used in models involves the physical form of the reactant. If a decay constant is applied to fluid concentrations and no other chemical interactions are allowed, then incorporation of the constant into the subroutine that computes fluid concentrations will not cause errors. If the situation being modeled involves chemical interactions such as precipitation,

ion-exchange, or sorption, which temporarily remove the radionuclide from solution, then it is important to use a second subroutine to account for the solid phase decay of the radionuclide (as done in a complex numerical model developed by Voss, 1984). Otherwise, the mass balance for the total radionuclide in the system will be incorrect. Depending on the relative amount of the contaminant on the solids, the mass balance errors could be quite serious; this would be true of organic chemical contaminants that heavily sorb, for example.

2.3.3 Chemical Associations

The establishment of new associations with other chemical species is not as well understood as the preceding interactions. Chemical associations nominally include ion-exchange, complexation, and co-solvation. Detailed understandings of these interactions are difficult to arrive at because of their nonspecific nature, characterized by a lack of the definite proportion of reactants to products (stoichiometry) that is typical of redox reactions. Through experiments with simple systems (e.g., ion-exchange of certain contaminants on specific resins or pure mineral surfaces), the general principles and driving mechanisms by which these interactions occur are known. The complex subsurface matrix in which they occur, however, provides many possible outcomes and renders predictions uncertain.

Ion-exchange reactions heavily influence the mobility of metals and other ionic species in the subsurface. For example, metal ions with a larger hydrated radius tend to be displaced by metal ions with smaller hydrated radii in most clays (Stumm and Morgan, 1981):

$$Na^+R^- + K^+ = K^+R^- + Na^+$$

The ionic charge density also exerts an influence in determining the relative affinities of metal ions for charged surfaces of solids, so that displacement of monovalent ions with divalent ions is also preferred (Tchobanoglous and Shroeder, 1985):

$$2Na^{+}R^{-} + Ca^{2+} = Ca^{2+}R_{2}^{2-} + 2Na^{+}$$

The influence of ion-exchange on organic contaminant transport, however, is not well understood. The strong-acid cation exchange that typifies the influence of metals is unlikely to take place with most organics, due to their weak ionization and polarity properties.

Complexation is yet another form of chemical association that is potentially important in contaminant transport. Based on studies of pesticides and other complex organic molecules, natural organic matter (such as humic and fulvic materials) can complex and thereby enhance the apparent solubility and mobility of synthetic organic chemicals (Carringer and others, 1975). The exact mechanisms by which humic and fulvic moities bind organics is difficult to determine experimentally. There appear to be a variety of weak bonding mechanisms involved, such as hydrogen bonding, π-electron bonding, and hydrophobic interactions (Wershaw, 1986). Research is needed to define the magnitude of such interactions, not only with naturally occurring organic molecules but also with other man-made organics present in contaminated environments. Research is also needed to determine if these complexes are stable and liable to transport through the subsurface. Examination of the degree to which synthetic organic chemicals complex toxic metals is needed, too. There is no theoretical objection to such interactions (indeed, certain metal-organic complexes are produced industrially, such as Na-EDTA /

sodium ethylene-diamine-tetra-acetic acid), and there is ample evidence that metals are moving through the subsurface at many waste sites.

Co-solvation occurs when a solvent is dissolved in the aqueous fluid phase (e.g., ground water) at concentrations that enhance the solubility of another contaminant. This occurs in agricultural uses, for example, where highly insoluble pesticides and herbicides are mixed with organic solvents to increase their solubility in water prior to field application. There is every reason to expect similar behavior at hazardous waste sites, where a variety of solvents are typically available. At present, prediction of the extent of the solubility increases that might occur at disposal sites in a complex mixture of ground water and several organic solvents is not possible.

Researchers have started examining co-solvation by working on relatively simple mixed solvent systems. Findings to date indicate that the sorption coefficients for hydrophobic organic chemicals decrease log-linearly with increasing fraction of organic co-solvent in binary solvent systems, and this observed behavior has been used to successfully predict the behavior of ternary solvent systems (Woodburn and others, 1986). This means that evidence has been found to support the idea that co-solvation can dramatically increase the overall solubility of hydrophobic organic contaminants (e.g., chloroform, benzene, trichloroethene); and that the solubility increases are predictable for systems containing only a few solvents. Further research in this area will be extremely useful, even if the results are limited to a qualitative appreciation for the magnitude of the potential effects of co-solvation. In particular, the enhanced contaminant mobility that accompanies solubility increases caused by co-solvation needs to be appraised.

At the extreme, the presence of organic solvents in the subsurface may result in a fluid phase that is separate from the aqueous fluid phase. In such cases contaminants are transported by two related mechanisms; coherent movement of the organic fluid phase and movement of that part of the aqueous phase that contains dissolved organic contaminants. Contaminants in separate organic fluid phases may move at a rate that is greater than or less than their mobility in the aqueous fluid phase. As the contaminants move with the organic fluid phase, they are released into adjacent aqueous fluids to a degree roughly proportional to their octanol-water partition coefficients. Laboratory determinations of the octanol-water partition coefficients are possible for most organic chemicals.

By releasing a portion of the organic phase constituents into adjacent ground waters, the partitioning process increases the total volume of subsurface material affected by contaminants. This is an important clean-up consideration. In studying the potential value of displacing spilled crude oil with solvents like cyclohexane, Alharthi and coworkers (1986) noted that the net volume occupied by the two immiscible fluids after water flushing remained relatively constant; only the application of surfactants and water-miscible organics improved the degree of displacement by water. They also found the displacement of immiscible fluids by water to be heavily dependent on the water-wetness of the subsurface materials just prior to innundation by spilled crude oil.

2.3.4 Surface Interactions

Of those interactions that involve organic chemicals in the environment, none has been as exhaustively studied as sorption (Boucher and Lee, 1972; Carringer and others, 1975; Karickhoff and others, 1979; O'Conner and Connolly, 1980; DiToro and Horzempa, 1982; Miller and Weber, 1984 and 1986; Mackay and others, 1986; Woodburn and others, 1986). Sorption studies relate the amount of contaminant in solution to the amount associated with the solids. A general expression for sorption is given by the Freundlich isotherm (Tchobanoglous and Shroeder, 1985; Tinsley, 1979):

$$x/m = K_F * C_e^{1/n}$$

where: x/m is the mass of contaminant sorbed to the mass of solids at equilibrium (µg/g),

K_F is the Freundlich equilibrium sorption constant,

C_e is the solution concentration at equilibrium (µg/L), and

$1/n$ is an experimentally derived exponent expressing the degree of non-linearity of the isotherm.

The contaminant is partitioned between the aquifer solids and the aqueous phase at ratios that are independent of (linear isotherms) or dependent on (non-linear isotherms) the equilibrium solution concentration. Sorption isotherms for nonpolar organic compounds commonly diverge from linearity well before the contaminant aqueous solubility becomes a consideration (Mackay and others, 1986). Coefficients derived from sorption isotherms can be used to approximate contaminant retardation in the subsurface. In fairly homogenous aquifers, retardation of relatively mobile contaminants in the field can be reasonably predicted from batch sorption isotherm studies. In more complex systems, sorption kinetics may cause deviations from batch observations, especially when high contaminant concentrations force sorption isotherms into a non-linear response.

Most often the sorption term in transport models is estimated for simplicity from the assumption that the response is linear (Miller and Weber, 1984). When linearity is assumed, the Freundlich equilibrium relationship approximates a 'distribution coefficient' (Freeze and Cherry, 1979; Anderson, 1979):

$$x/m = K_d * C_e \quad \text{or} \quad x/m \,/\, C_e = K_d$$

where: K_d is the distribution coefficient, which is used to estimate an apparent retardation factor by:

$$R = V \,/\, V_c = 1 + (p_b \,/\, \emptyset_e) * K_d$$

where: R is the retardation factor (dimensionless),

p_b is the bulk density of the aquifer, and

V_c is the apparent contaminant velocity.

The left-hand side of the advection-dispersion equation is multiplied by the retardation factor to approximate the effects of sorption on contaminant transport (Anderson, 1979; Bear, 1979; Miller and Weber, 1984):

$$[(D_x * (\delta^2C/\delta x^2)) + (D_y * (\delta^2C/\delta y^2)) + (D_z * (\delta^2C/\delta z^2))]$$
$$- [(V_x * (\delta C/\delta x)) + (V_y * (\delta C/\delta y)) + (V_z * (\delta C/\delta z))]$$
$$= \delta C/\delta t * (1 + (p_b \,/\, \emptyset_e) * K_d) \qquad = \delta C/\delta t * R$$

This approximation can produce serious mass balance errors. When using a linear assumption for predicting the behavior of a contaminant whose true Freundlich exponent is greater than one, the solution phase concentrations will be over-estimated. They will be under-estimated if the true Freundlich exponent is less than one (Figure 2.2). For example, Lee and co-workers (1986) found that use of linear adsorption coefficients for fenamiphos and its metabolites would result in underprediction of the amount sorbed by about two-fold, especially with low levels of contaminants in solution. In practical applications, this means that their linearization wuold result in fenamiphos not being detected at a monitoring well until long after it was predicted to.

The opposite kind of effect, a contaminant arriving before it is expected, is also quite possible when linearizing a sorption relationship. Consider the following hypothetical example using Figure 2.2. Assume that a particular contaminant in contact with a certain loamy soil has been shown to yield an isotherm similar to the one in Figure 2.2 with a Freundlich exponent of 0.7, and that the solution concentration obtained from a shallow farm well is 10 µg/L. The true isotherm relationship would predict there to be about 5 µg/g sorbed to the soil; but a linearized version (the line with the exponent equal to 1.0) would predict that 10 µg/g is sorbed to the aquifer solids -- roughly overestimating the degree of sorption by 100%.

To resolve the discrepancy between predicted and actual transport in such cases, most practitioners would arbitrarily adjust some other poorly-characterized model parameter; for example, dispersion. This leads to the creation of a model that does not present the various natural

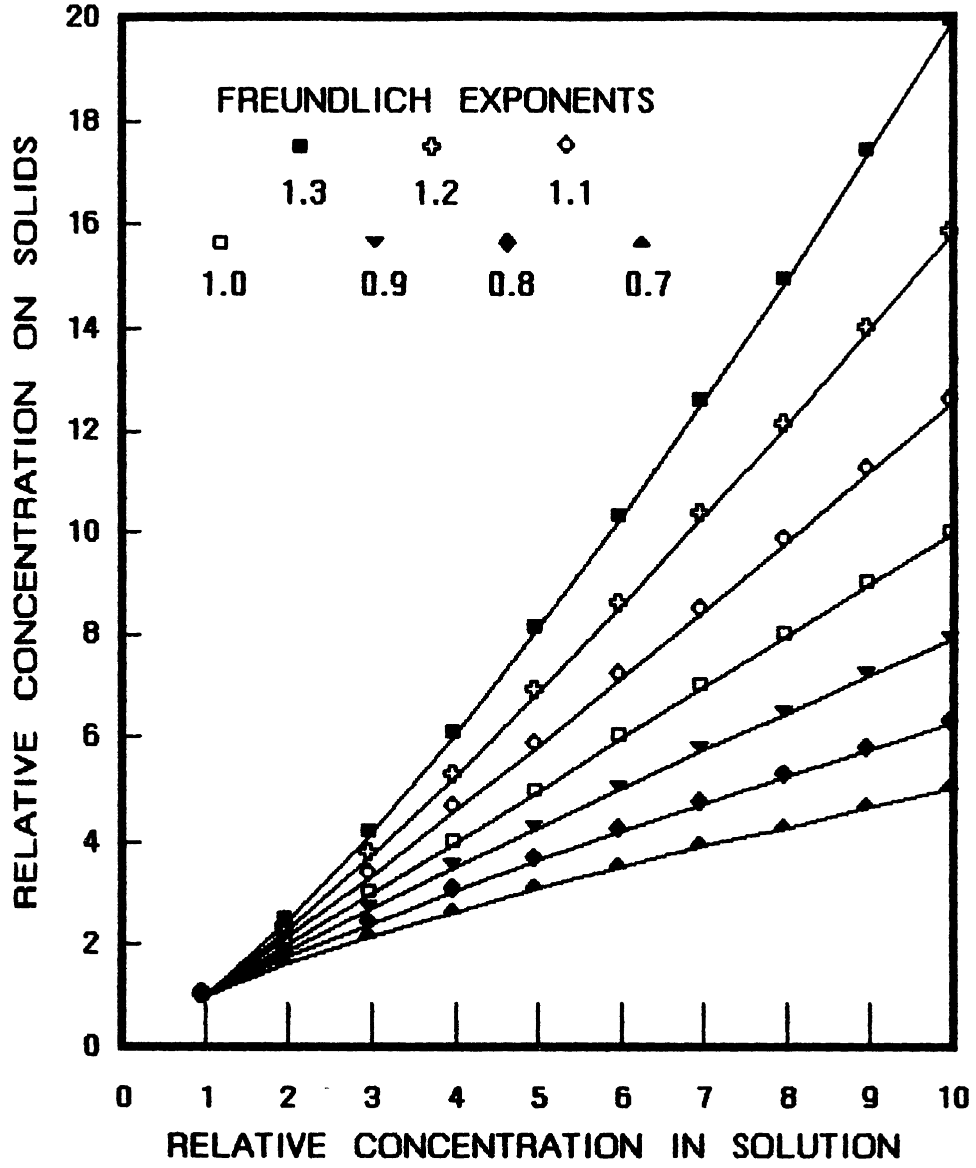


Figure 2.2. Examples of Freundlich Isotherms with Various Exponents

process influences in proper perspective. The predictions from such models are likely to be qualitatively, as well as quantitatively, incorrect. More widespread consideration should be given to accurate representation of non-linear sorption, particularly in transport modeling at contaminated sites.

The time dependency of the sorption process is a related phenomenon that has recieved little attention in practical applications of sorption theory. Most models assume that sorption is instantaneous and completely reversible (local equilibrium). A growing body of evidence argues to the contrary, especially for large organic molecules in high-carbon soils, such as pesticides in agricultural soils (Miller and Weber, 1986). Mineral surfaces appear to control soprtion behavior in low-carbon aquifer materials (e.g., studies of tetrachloroethene in a sandy aquifer reported by Mackay and others, 1986). Sorption and desorption kinetics are undoubtedly important in contaminant transport, but the implications have yet to be systematically defined.

For example, to what degree kinetically slow release of a contaminant would affect pump-and-treat remediations is a topic worthy of immediate investigation. Due to the increased flow velocities caused by pumping, the length of time that ground waters are in contact with contaminated soils or sediments may be less than the time required to reach maximum equilibrium concentrations. That would mean that pumpage would have to continue for much longer periods to exhaust all of the contaminant reserves sorbed to the aquifer solids. It would also mean that relatively low concentration wastewaters would be produced by the pumping wells, which would increase treatment costs since air-stripping and activated carbon treatment efficiencies drop with lower concentrations.

Evidence from Superfund sites and ongoing research activities suggests that strong association with solid surfaces does not preclude contaminant mobility, because of the potential for particle movement alluded to earlier. In many instances, especially in glacial and alluvial sediments that contain a wide distribution of particle sizes, substantial accumulations of fine aquifer materials have been found in the bottom of monitoring wells. In addition, iron-based colloids have been identified in ground waters downgradient from a site contaminated with domestic wastewater. If contaminants can associate with fine particles such as these, their mobility through the subsurface could be markedly enhanced. To determine the significance of particle transport in the movement of highly retarded contaminants, studies of actual materials from contaminated sites would be very helpful.

Although knowledge about the foregoing chemical processes and their functions in the subsurface has been significantly improved upon in recent years, this information is only slowly finding its way into practical interpretations of pollutant transport at contaminated sites. Evidence from field sites suggests that much remains to be learned about these processes.

2.4 Biological Processes

Many contaminants that enter the subsurface environment are biologically reactive. Under appropriate circumstances they can be completely degraded to harmless products. Under other circumstances, however, they can be transformed to new substances that are more mobile or more toxic than the original contaminant (e.g., successive reductive biotransformation of tetrachloroethene to vinyl chloride). Quantitative predictions of the fate of biologically reactive substances are at present very primitive, particularly compared to other processes that affect pollutant transport and fate. In most respects, this situation seems to have resulted from too much borrowing of concepts from surface water models of biotransformation (e.g., the biofilm models discussed by Bouwer and McCarty, 1984). The ground-water community may have chosen inappropriate conceptual models as the basis for representations of subsurface biotransformations.

2.4.1 The Surface Water Model

Until recently ground-water scientists considered aquifers and soils below the zone of plant roots to be essentially devoid of organisms capable of transforming contaminants (McNabb and Dunlap,1975; Jenkinson and Ladd, 1981). This belief prevailed because microbial organisms were looked for in ground-water samples only; as opposed to population studies of organisms attached to solid surfaces in the subsurface. Recent studies have shown that water-table aquifers harbor appreciable numbers of metabolically active microorganisms (Wilson and others, 1983; Webster and others, 1985), and that these microorganisms frequently can degrade organic contaminants (Alexander, 1981; Criddle and others, 1986; Tsukano, 1986). Unfortunately many ground-water scientists have presumed that the conceptual model most frequently used to describe biotransformations in surface waters is also appropriate to biotransformations in the subsurface.

In the surface water modeling approach, the presence of a contaminant is assumed to have no effect on microorganism populations that degrade it (Stumm and Morgan, 1981; Tchobanoglous and Schroeder, 1985). It is also assumed that contaminant concentrations do not influence transformation kinetics, and that the capacity to transform the contaminant is uniformly distributed throughout the body of water under study. These assumptions are probably appropriate for most surface waters: contaminant concentrations are usually too low and the residence times too short to allow adaptation of the microbial community to the contaminant, and the organisms that are naturally pre-adapted to the contaminant are mixed throughout the water body by turbulence. Consequently, utilization kinetics can be described by simple first-order decay constants. In surface waters these constants are usually obtained by monitoring contaminant disappearance over time in replicated water samples.

2.4.2 Ground-water Biotransformations

Circumstances such as those described for surface waters rarely apply to biotransformations in the subsurface. Contaminant residence times are usually long, at least weeks or months, and frequently years or decades. Further, contaminant concentrations that are high enough to be of environmental concern are often high enough to elicit adaptation by the microbial community (Tinsley, 1979; Alexander, 1981; Criddle and others, 1986). For example, the U.S. Environmental Protection Agency's Maximum Contaminant Level (MCL) for benzene is 5 ug/L. This is very close to the concentration of alkylbenzenes required to elicit adaptation to this class of organic compounds in soils. As a result, the biotransformation rate of a contaminant in the subsurface environment is not constant, but increases after exposure to the contaminant in an unpredictable way. Field work has shown that the transformation rate of organic contaminants in aquifers, such as alkylbenzenes, can vary as much as two orders of magnitude over a meter vertically and a few meters horizontally (Wilson and others, 1983; Bedient and others, 1984; Webster and others, 1985). This surprising variability in transformation rates is not related in any simple way to system geology or hydrology.

It is difficult to determine first-order rate constants in subsurface materials. Most microbes in subsurface materials are firmly attached to solid surfaces; usually less than 1% of the total population is truly planktonic. As a result, the microbes in a ground-water sample grossly underrepresent the total microbial population in the aquifer. Thus, contaminant disappearance kinetics in a ground-water sample do not represent the behavior of the material in the aquifer. It is therefore necessary to perform microcosm studies with samples representative of the entire aquifer system; and that is a formidable technical challenge.

2.4.3 A Ground-water Model

These findings have prompted re-examination of assumptions about biotransformation implicit or explicit to a particular modeling approach, with the realization that no one qualitative description of biotransformation can be universally applicable. Field experience has shown that the expressions that describe the biological fate of contaminants actually change within aquifers in response to geochemical constraints on microbial physiology. Rather than describing biotransformation with a continuous function applicable at all points in the aquifer, it may be more realistic to examine key geochemical parameters and to use that information to identify the relationship for biotransformation that applies at any particular point. These key parameters could include the contaminant concentration, oxygen or other electron-acceptor concentrations, redox state, pH, toxicity of the contaminant or co-occurring materials, and temperature. One such model has been evaluated in the field (Bedient and others, 1984).

The model proposed by Bedient and his colleagues described an alkylbenzene and polynuclear aromatic hydrocarbon plume in a shallow water-table aquifer. Microcosm studies showed that organisms in the aquifer had adapted to these contaminants, and would degrade them very rapidly when oxygen was available. As a result of this adaptation, the hydrocarbon biodegradation rate was not controlled by any inherent property of the organisms. Rather, the physical transport processes of diffusion and dispersion seemed to dominate by controlling oxygen availability to the plume. Because the biotransformation rate was controlled by physical processes, the actual model was very simple. Oxygen and hydrocarbon transport were simulated as conservative solutes using a popular two-dimensional numerical model, the U.S. Geological Survey's method-of-characteristics code (Konikow and Brederhoeft, 1978). A special subroutine was added to the USGS code that would examine oxygen and hydrocarbon concentrations at each node in the model grid so as to generate new concentrations based on oxidative metabolism stoichiometry. When the model was used to project plume behavior at future times, it illustrated an important property of many such plumes. The plume continued to spread until the rate of admixture of oxygen balanced the rate of release of hydrocarbons from the source. Afterward, the extent of the plume remained relatively constant.

The body of field experience which can be drawn on to assign laws for biotransformation is growing rapidly. Transport-limited kinetics may commonly apply to releases of petroleum hydrocarbons and other easily degradable materials such as ethanol or acetone in oxygenated ground waters. On the other hand, materials that can support a fermentation, in which an exogenous electron acceptor is not required, may follow first-order kinetics. Unfortunately, many important biotransformations in ground water are still mysteries. The reductive dehalogenation of small halogenated hydrocarbons such as trichloroethene and 1,1,1-trichloroethane is a good example. In such cases transformation kinetics of the compound are controlled by transformation kinetics of a second compound, the primary substrate that supports the metabolism of the active microorganisms (Bouwer and McCarty, 1984). These complex interactions are poorly understood and cannot be described quantitatively at the present time. However, this is an area of active research, and hopefully the appropriate relationships will soon be determined.

CHAPTER III

DATA ACQUISITION AND INTERPRETATION METHODS

3.1 Monitoring Well Networks

Monitoring wells are essential elements in the characterization of ground-water contamination. In designing networks of monitoring wells, consideration needs to be given to the number of monitoring wells, their locations, the methods by which they are constructed and sampled, and their sampling frequency. Choices in the design of monitoring well networks are also influenced by properties of the contaminant(s) and the hydrological conditions. The chosen design should maximize chances for detection of contamination and should obtain the most information for the least cost. These are not independent factors, but are interrelated.

3.1.1 Hydrogeologic Considerations

Ground water preferentially flows through the most coarse, most permeable material. The more permeable the material, the greater the proportion of flow passing through the material. The magnitude of the difference between flow rates in adjacent low and high permeability materials is staggering to contemplate. Typically, the contrast between coefficients of permeability for low permeability deposits (e.g., clays) and high permeability deposits (e.g., coarse sands and gravels) is two to six orders of magnitude. This means that the spatial variability of permeability will cause ground water to flow through intricate pathways not readily expressed by average hydraulic gradients over moderate distances (Figure 3.1). If the ground-water flowlines are not perpendicular to the water level contours, as occurs in anisotropic aquifers, then the task of predicting where contaminants will migrate becomes increasingly difficult. This is because it is considerably more economical and technically feasible to measure water levels than it is to measure actual flow paths, such as is attempted with tracer experiments.

Other reasons why contaminants may not travel along the same flowlines as native ground waters relate to immiscible and dense plume behavior. Immiscible plumes, such as petroleum product spills, may float on top of the water table (e.g., gasoline) or sink to the bottom of the aquifer without appreciable mixing (e.g., crude oil). Miscible plumes of significantly higher density than native ground waters (e.g., brines and salt water) usually migrate downward through aquifers, with only slow mixing and minimal horizontal movement. Moreover, the horizontal movement of dense plumes may oppose or crosscut the direction in which native ground waters flow since dense plumes migrate by gravity flow along the slope of impermeable strata or bedrock lying below unconsolidated deposits (Mackay and others, 1985). Hence, the design of monitoring well networks to detect such plumes becomes an almost intractable problem.

The remainder of this section will focus on the design of monitoring well networks to detect miscible plumes having densities similar to the ground waters they contact. Limiting the discussion in this way is not unreasonable, given the vast number of contaminant plumes falling into

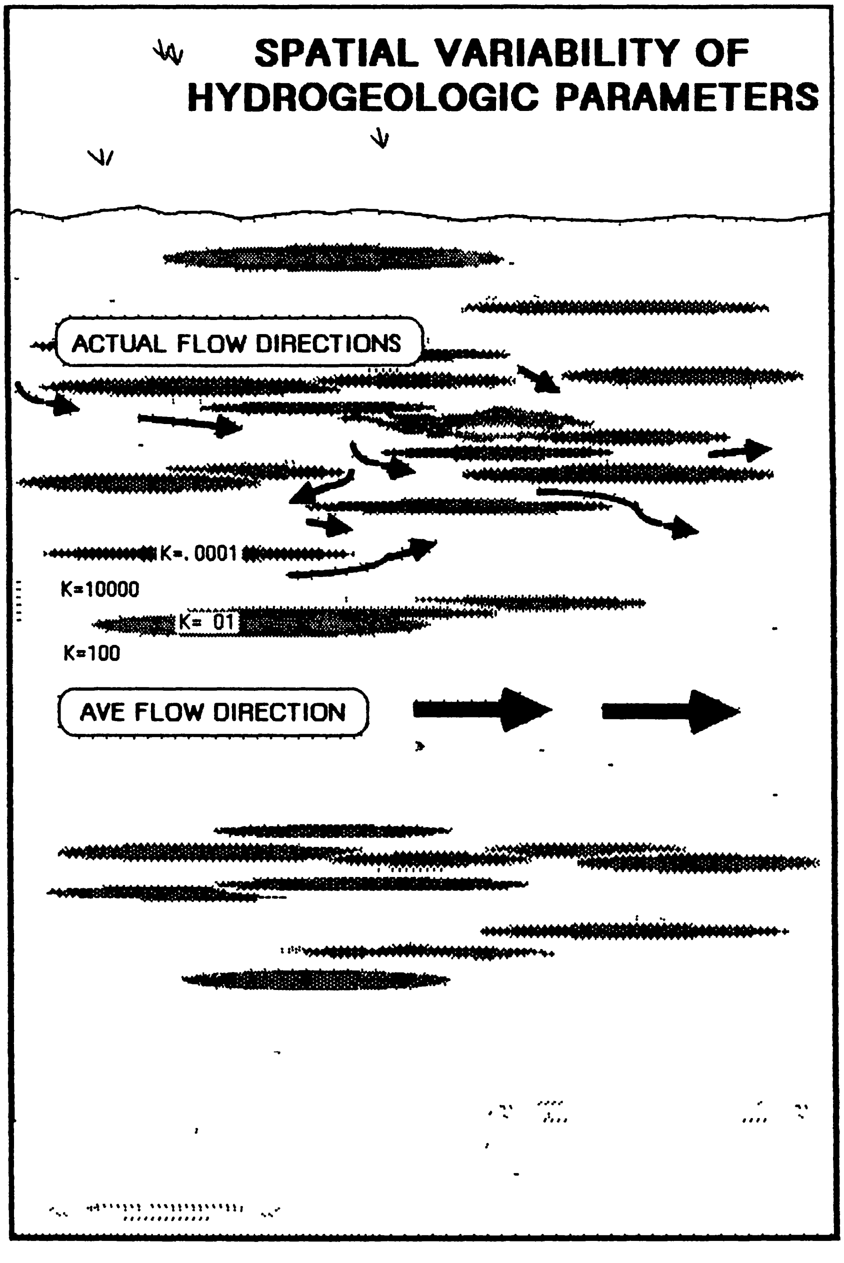


Figure 3.1. Flowpath Complications Caused by Spatial Variability of Coefficient of Permeability in Heterogeneous Strata

this category. Many of the points made here will be useful for contamination problems involving immiscible or dense plumes, too. Often the most troublesome issues associated with those plumes involve the co-occurrence of highly mobile secondary plumes, such as the trace organic chemical contaminants that partition into ground water at interfaces with petroleum spills.

3.1.2 Contaminant Sources

Contaminants most often enter the aquifer from the ground surface by infiltration and subsequent percolation downward through the unsaturated zone; though direct contact with shallow water tables and direct injection through disposal wells are appreciable sources, too. Contaminant sources may be continuous or discontinuous ('slug' releases). Landfills, waste storage sites, pits, ponds, and lagoons are commonly continuous sources. Spills, underground storage tanks, and pipelines are typical intermittent or discontinuous sources.

Continuous releases from contaminant sources usually generate steep concentration greadients near the source; contaminant levels may range over four orders of magnitude within hundreds of feet of a continuous source. Slug releases are inherently more difficult to detect than continuous releases of contaminants because of their smaller sizes generally, and because ot their limited duration of exposure at any given point. Peak contaminant concentrations in slug-release plumes are continuously moderated by dispersion, retardation, and transformation processes; the range of contaminant levels may be only an order of magnitude or two by the time the plume has migrated downgradient a thousand feet or more.

Once released, contaminants are transported by ground water toward discharge points. The extent of diffusion and dispersion which may occur depends upon the actual trajectory of ground-water flowlines. Non-conservative contaminants will interact with other contaminants or with the solid phase of aquifers, resulting in retardation of the contaminant compared to the natural ground-water velocity. For routine monitoring one is usually interested in those contaminants that travel at or near the same velocity as water, such as chloride, nitrate, and many volatile organic chemicals. Near pumping wells ground-water flowlines converge. This results in a contraction of the width of the plume and an increase in its migration rate, making it harder to detect and intercept. Hence, choosing the optimal locations for monitoring wells near production wellfields is difficult.

Given these comments, and the preceding section's synopsis of factors which govern the transport and fate of contaminants, choices for the locations and sampling frequencies for monitoring wells can be examined. In conducting such examinations it is instructive to think of the problem as having three zones of interest: a near-source zone, a transition zone, and a near-wellfield zone (Figure 3.2).

3.1.3 The Near-Source Zone

Close to the source of the contaminants special attention must be paid to the spatial and temporal characteristics of the release. Spatially, one must recognize that some sources (e.g., lined landfills and waste storage lagoons) often release contaminated fluids at fairly low rates, whereas other sources (e.g., unlined liquid storage lagoons,

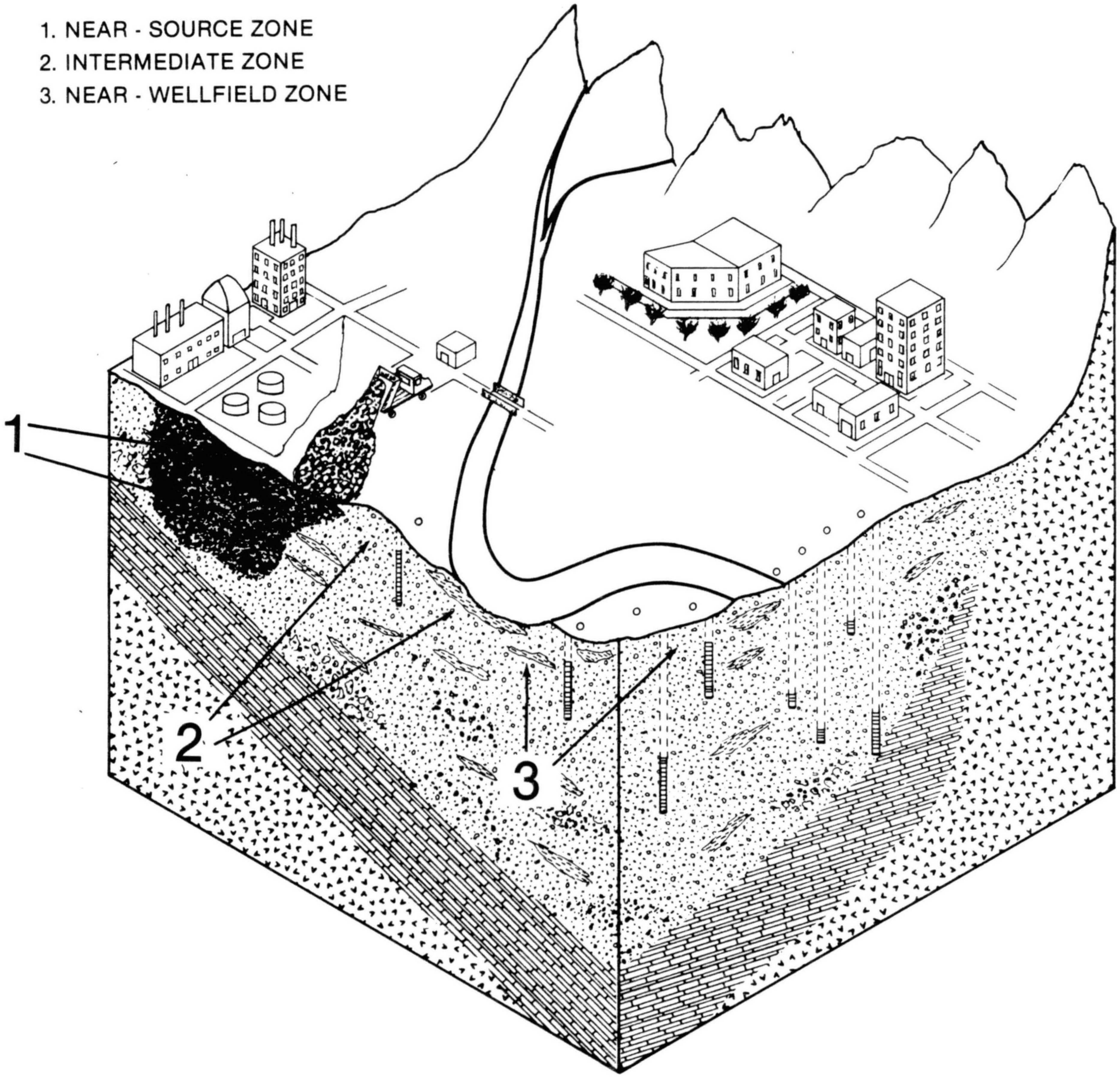


Figure 3.2. Typical Ground-Water Contamination Scenario with Special Zones of Interest Indicated

and waste disposal wells) may release contaminated fluids at high rates. The greater the rate of release, the more significant becomes an accompanying distortion in the natural shape of the water-level contours. The primary result of these spatial distortions is a marked increase in ground-water velocities locally.

Temporally, one must recognize the intermittent nature of many source releases. If a source releases contaminants continuously, the ability to detect contaminants with only minimal sampling of monitoring wells is statistically much greater than is the case for intermittent releases. This is particularly true for low-rate source releases; whereas the more rapid movement of contaminant released from sources at high volumetric rates necessitates frequent sampling of monitoring wells. Obvioiusly, the most difficult kind of release to detect is that from a high-rate, intermittent source. These concepts are illustrated in Figure 3.3.

As a practical matter, it is suggested that the design of a monitoring well network begin with a survey of potential sources of contaminants. When they have been identified and categorized according to their probable spatial-temporal contaminant release characteristics, it will be possible to recommend appropriate locations and sampling frequencies for monitoring wells. Table 3.1 presents some suggested guidelines for the various near-source situations discussed here.

3.1.4 The Intermediate Zone

Because this zone does not include major fluid sources or sinks, by definition, contaminant movement in the intermediate zone will occur at

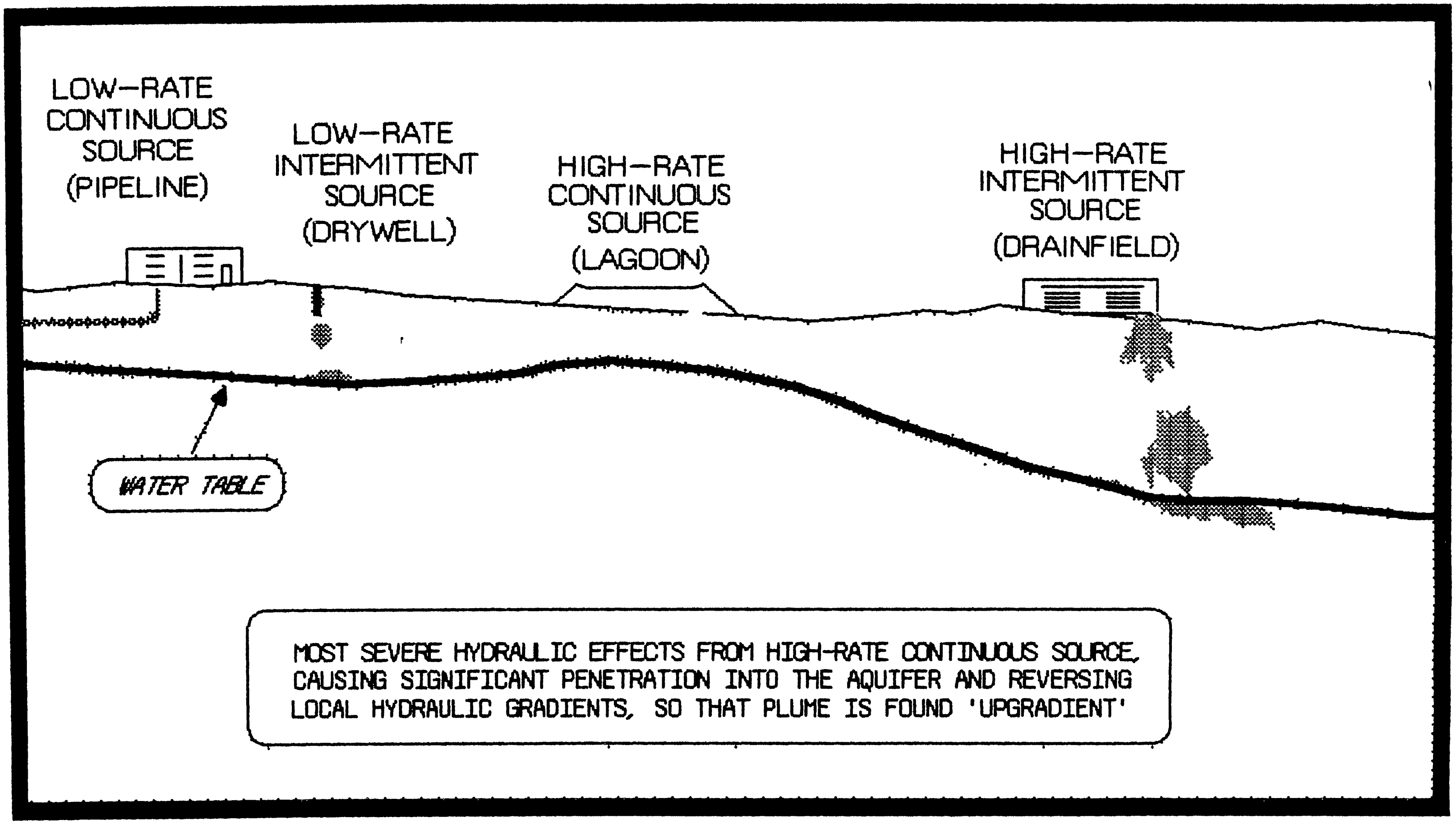


Figure 3.3. Various Contaminant Source Release Possibilities

Table 3.1 Monitoring Strategies Near Contaminant Sources

* Potential sources can be identified by review of historical photographs and zoning plats, and interviews with local citizens.

* The nature of wastes produced by a potential source should be ascertained by visual inspection, review of existing data and historical documents, and interviews with site owners and operators.

* Clusters of vertically-separated monitoring wells should be installed close (e.g., 50 to 100 ft.) to the contaminant source.

* Each cluster should contain enough monitoring wells to enable estimates of vertical gradients and chemical profiles to be made, with additional wells needed in aquifer recharge and discharge areas and in multiple-aquifer settings.

* At least three clusters should be used; many more if the water surface is expected to be significantly non-planar (e.g., ground-water mounds beneath leaking lagoons).

* The locations of the clusters should not be restricted to areas known to be regionally downgradient, as local gradients may be dominated by the source.

* On a quarterly basis, ground-water samples should be collected from the monitoring wells and analyzed for a select list of chemicals; the full spectrum of priority pollutants should be analyzed for annually.

* The list of chemicals to analyze quarterly samples for should include class representatives for all major natural and synthetic chemical groups (e.g., cadmium, lindane, pentachlorophenol, tetrachloroethene, etc.) with emphasis on the most mobile contaminants and those known to be released from the source.

* Water-level measurements should be obtained quarterly.

or below the rate of the natural ground-water flow. Unless contamination has been detected just upgradient, there is usually little or no need to place monitoring wells in the intermediate zone. Rather, it is sufficient to sample existing wells (e.g., at private homes, farms, industry) on an annual or biannual schedule.

Of course, this advice should be tempered with some caution if there is great uncertainty regarding activities that may occur in the intermediate zone (e.g., large numbers of homes on individual septic systems, nitrates and pesticides associated with farming, onsite waste storage or disposal by industries). In other words, one must establish the probable lack of existence of potential contaminant sources prior to adopting a low-intensity sampling schedule for an intermediate zone.

3.1.5 The Near-Wellfield Zone

One of the key concepts involved when assessing the potential for contamination of a water-supply wellfield is that of the 'catchment area', which is that portion of the recharge zone of the aquifer that is actually available to a specific well or wellfield. Each production well or wellfield (cluster of production wells) has a uniquely configured catchment area. In a water-table aquifer the catchment area has the same bounds as the 'capture zone' (which separates waters flowing into the well or wellfield from those in the rest of the aquifer - to be further defined momentarily); recharge is often largely the result of local infiltration of rainwater. If there is little or no regional ground-water flow this will approximate a circular area (Figure 3.4). If there is an appreciable ground-water flow rate, the capture zone and

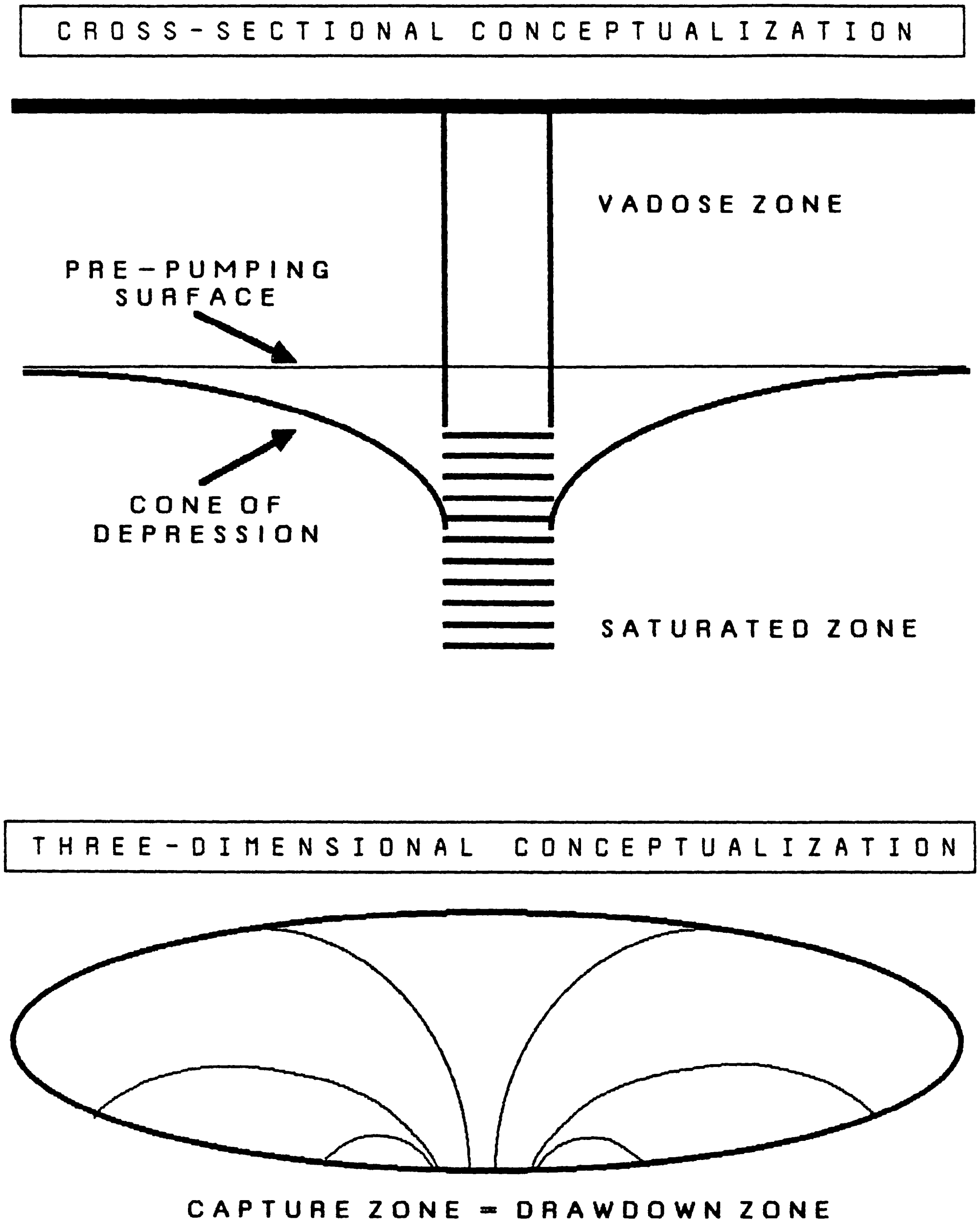


Figure 3.4. Capture Zone of a Pumping Well in a Water Table Aquifer with Negligible Regional Flow

catchment area will extend upgradient to the limits of the aquifer recharge zone; but will be truncated downgradient (Figure 3.5). The catchment areas of semi-confined and artesian aquifers are considerably more removed from the vicinity of the pumping well than is true of water-table aquifers; whereas the capture zone is a localized phenomena for all cases. The primary recharge mechanism for fully confined aquifers is replenishment through an outcropping recharge zone at the upgradient limits of the aquifer (Figure 3.6). Recharge through overlying or underlying aquitards is important for semi-confined aquifers (Figure 3.7).

Dramatically increased ground-water velocities lie upgradient of each pumping well and a velocity divide lies downgradient (the situation is reversed for injection wells), and to the sides. Pumping a well has an additive effect on the flow velocity of fluids moving towards its upgradient side (e.g., the velocity from pumpage + the velocity of the natural flow = the net velocity upgradient). Conversely, pumpage causes the movement of fluids from the downgradient side of the well to oppose the natural flow velocity (e.g., the velocity from pumpage - the velocity of the natural flow = the net velocity toward the well from downgradient locations). As a result, at some distance downgradient there is an exact balancing between the tendency of the natural flow to move on downgradient, and the pull of waters back upgradient to a pumping well. This point is termed the 'stagnation' point because the ground-water flow velocity there is zero and no movement occurs (Figure 3.8).

The stagnation point is the downgradient boundary of what has been called the 'capture zone' (Keely and Tsang, 1983). The capture zone has

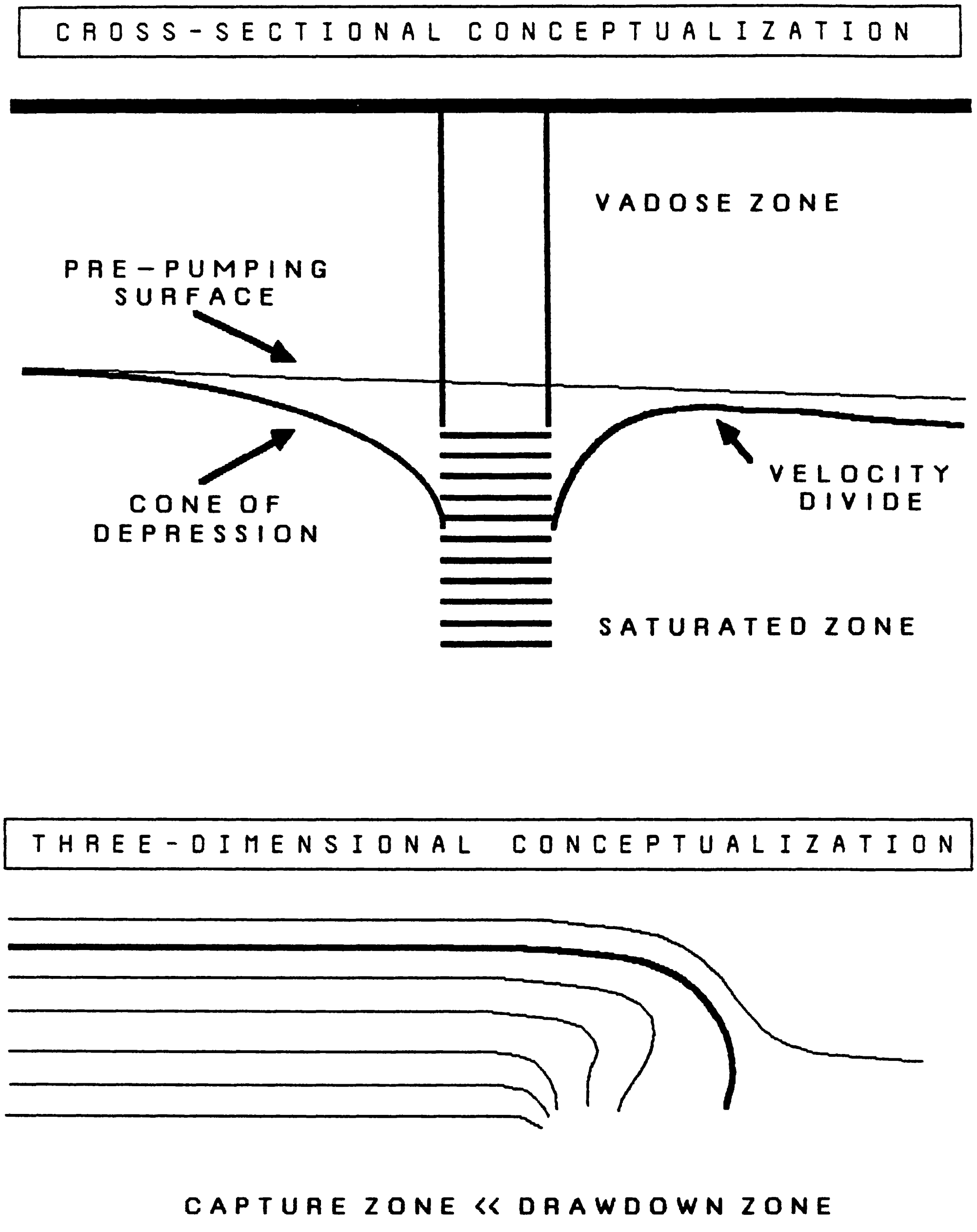


Figure 3.5. Capture Zone of a Pumping Well in a Water Table Aquifer with Appreciable Regional Flow

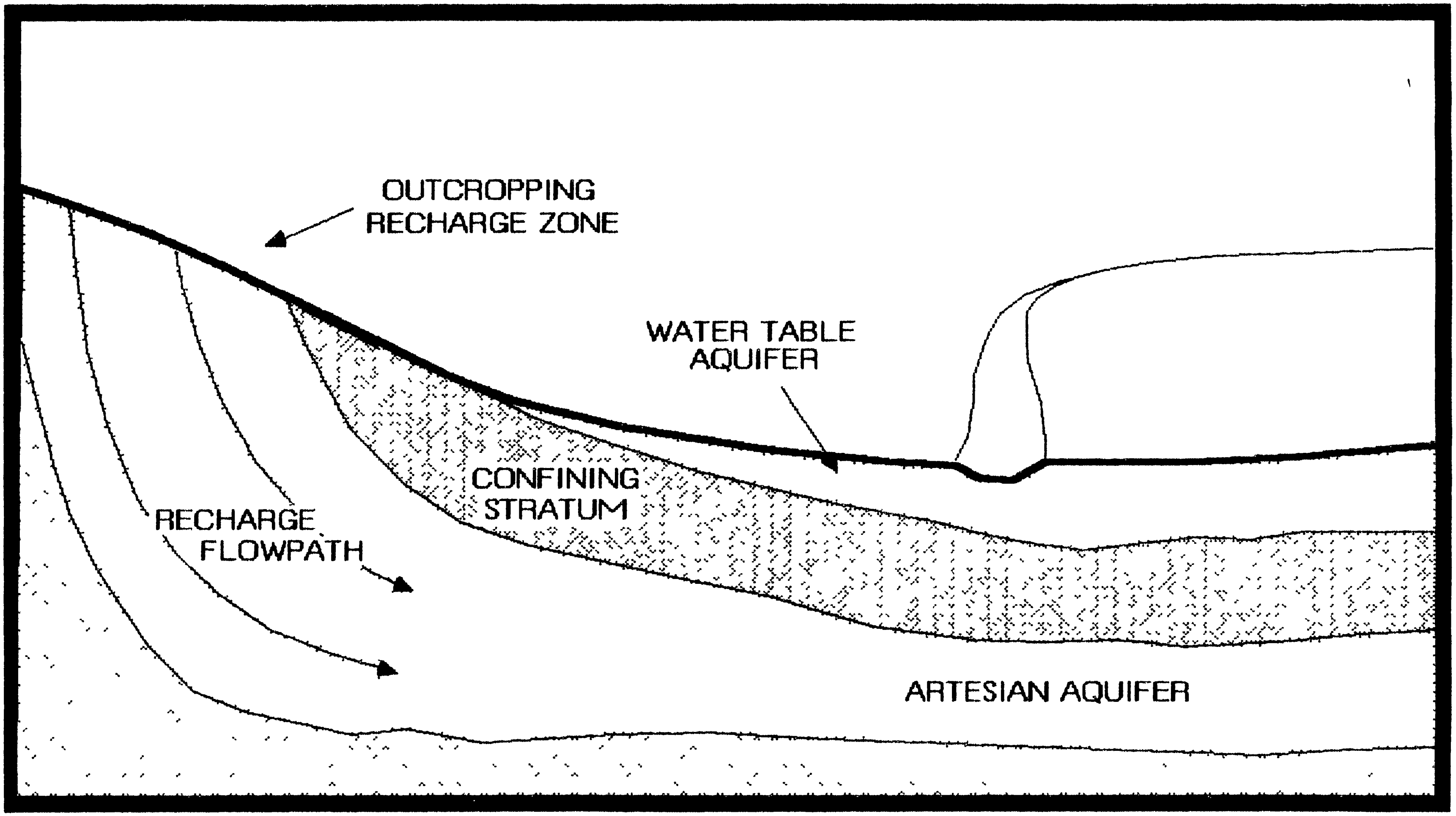


Figure 3.6. Recharge Mechanism for Fully Confined Aquifers

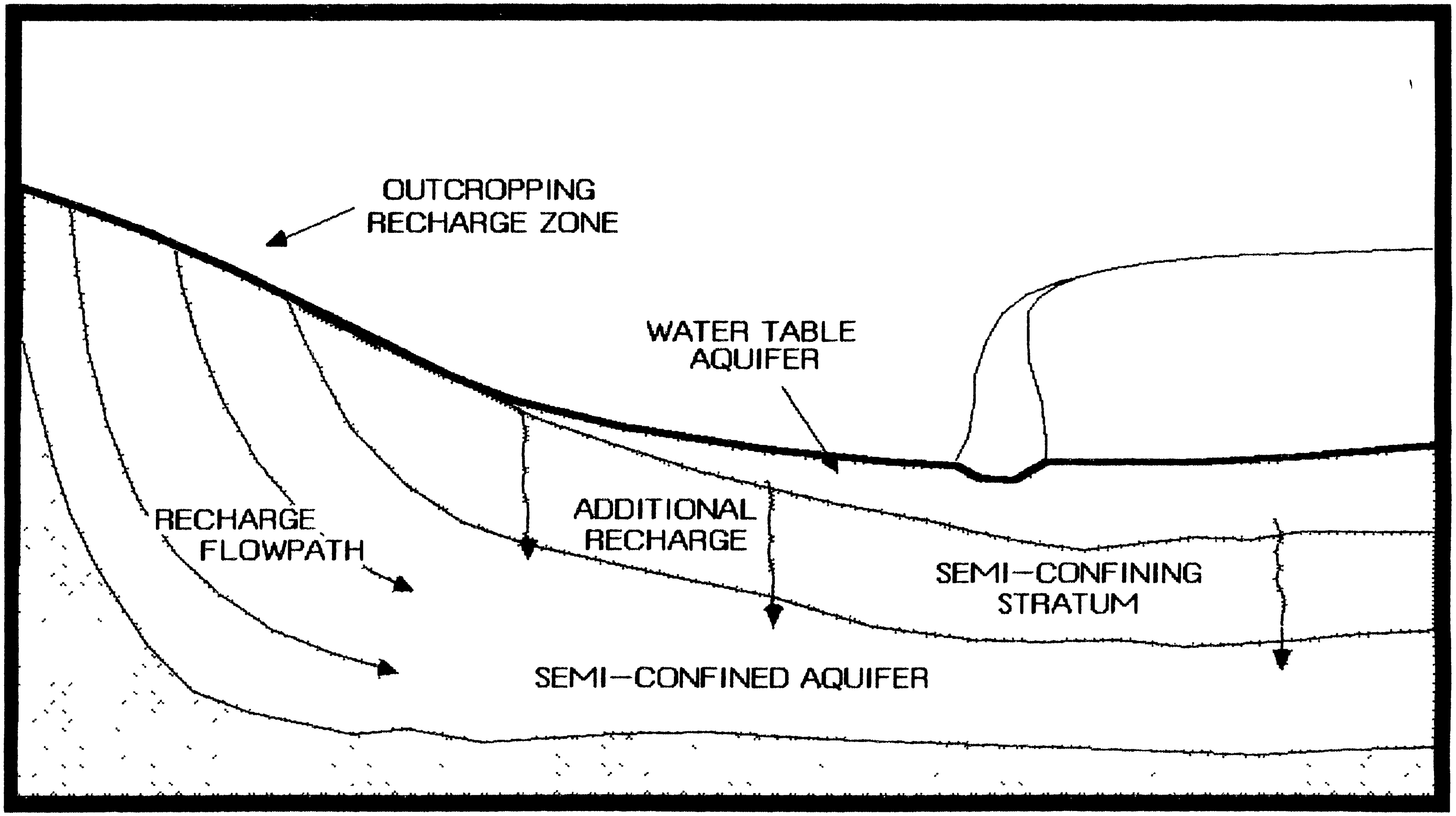
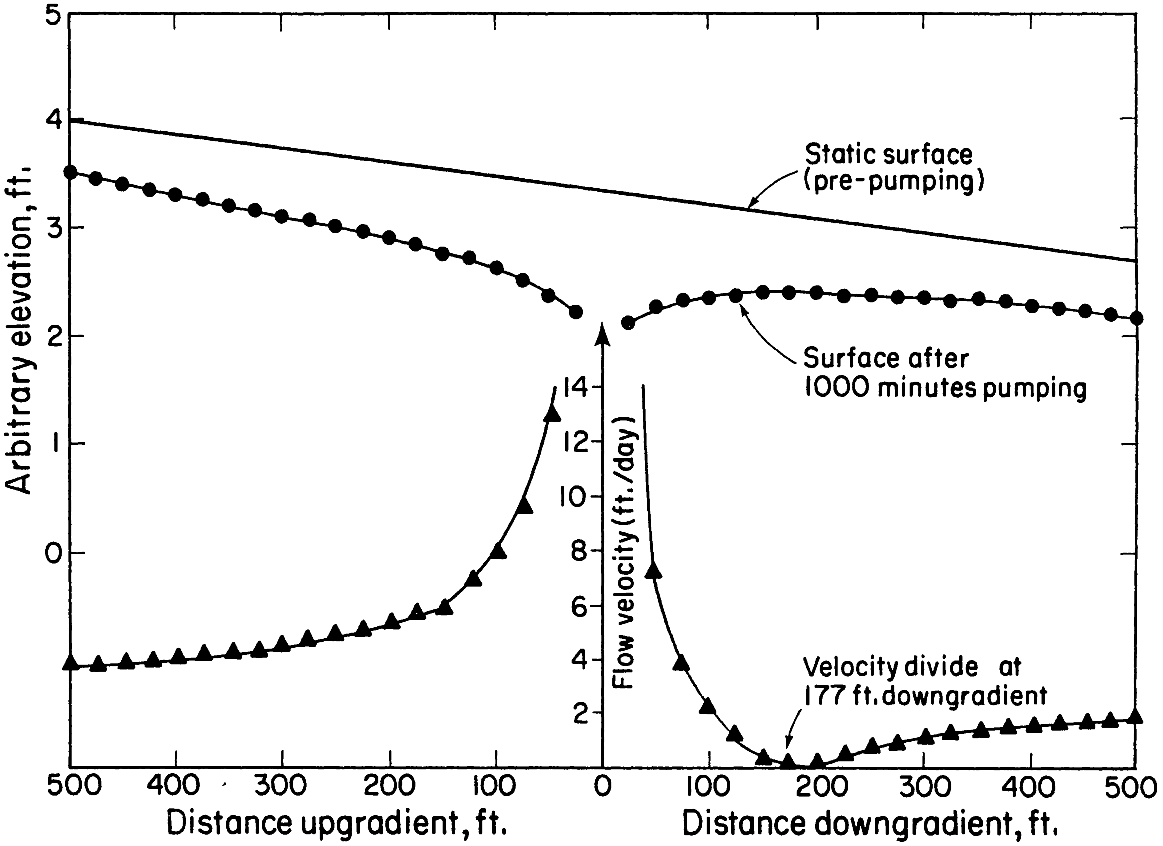


Figure 3.7. Recharge Mechanisms for Semi-Confined Aquifers



Notes: Plotted points assume that T = 6700 ft.^{2}/day, S = 0.005, h = 100 ft., K = 670 ft./day, ϕ_e = 25%, and I = 0.0013.

Figure 3.8. Cross-Sectional Velocity Field and Drawdown Characterizations of a Capture Zone

lateral boundaries that are at a distance of π (3.1416) times the nominal distance from the pumping well to the stagnation point. If two or more capture zones overlap, as would often occur in a wellfield, stagnation zones will form. These are of critical importance due to the extremely long periods of time which may elapse before any movement of fluids (and contaminants) occurs into, or out of, such zones. One can imagine that placement of a monitoring well in a stagnation zone will not be a good investment because the quality of stagnant waters there will change very little over time (assuming continuous operation of the production wells that create the stagnation zone).

It is clear that spatial and temporal variations in wellfield operations may cause significant distortions of the shape of a contaminant plume. Stagnation zones will exist during pumping, but will cease to exist when pumpage ceases. The size and shape of a stagnation zone depends upon the flowrates of the wells creating it. If those flowrates change, then the geometry of the stagnation zone also changes. Further, fluids which flow toward a well move along streamlines at velocities that are characteristic of their unique positions within the capture zone of the well. If one were to clock the time taken to move toward a well from a position upgradient near a lateral boundary of the capture zone, then it would be shown to take a far greater time to reach the well than would be the case for fluids moving along a streamline closer to the center of the capture zone.

In practice this means that monitoring wells should be placed some distance inside the lateral boundaries of the capture zone. It also means that there should be a moderate density of monitoring wells present, and that they should be sampled frequently (e.g., quarterly or semi-annually) so that prompt action can be taken. Table 3.2 presents some suggested guidelines for the various near-wellfield zone situations discussed here.

3.2 Monitoring Well Installation and Sampling

Some of the key problems that face the hydrogeologist involve choices in the installation and sampling of ground-water monitoring wells at hazardous waste sites. The selection of an installation method is often based on the expected time and costs, or on the ready availability of certain drilling equipment, rather than being tailored to hydrogeologic conditions at the site. This situation is regrettable, however, because the quality and representativeness of ground-water samples can be greatly affected by monitoring well installation and sampling techniques. The total funds expended to obtain and analyze ground-water samples over the lifetime of a monitoring well usually pale in comparison with monies spent on its installation. The potential monetary losses resulting from actions predicated on faulty data could be very high, as well.

There is no ideal monitoring well installation method for all purposes, so one should consider specific conditions at a site before deciding which drilling and development methods to use. The most widely used drilling methods include rotary methods, the cable tool or percussion method, and augering. Common development techniques include air-lift, surging and bailing, and overpumping. Specialized techniques for installation of monitoring wells at hazardous waste sites have begun to evolve from these conventional installation methods.

Table 3.2 Monitoring Strategies Near Wellfields

* As-built engineering plans and drillers logs should be examined to identify the strata open to the wellscreen(s).

* Accurate flowmeters should be installed on all pump discharge lines to ensure knowledge of total pumpage.

* Clusters of vertically-separated monitoring wells should be installed moderately close (e.g., within 500 ft.) to the well(s).

* Each cluster should contain a minimum of three monitoring wells, since vertical hydraulic gradients will be significant.

* More than three clusters should be used because the water level surface will be highly non-planar.

* The locations of the clusters should include downgradient, as well as upgradient, areas; all should lie within the capture zone of the well(s).

* Water level measurements should be obtained quarterly if the wellfield is in continuous operation, or much more often (e.g., weekly to monthly) if the wellfield is operated intermittently.

* Chemical analyses should performed as per recommendations for near-source areas (Table 3.1), except that samples from the production wells themselves should be included.

Prior to sampling a monitoring well, it is important to purge it of the stagnant waters in casing storage, so that the well contains only waters that have been freshly withdrawn from the aquifer. In wells having significant volumes of water in casing storage above the well screen, special measures must be taken to ensure removal of those waters (regardless of the number of borehole volumes purged, as will be discussed below). Purging also removes any sediment that may have accumulated between sampling events. The amount of accumulated sediment directly relates to the effectiveness of the gravel pack in filtering fine grained sediments.

The literature contains sufficient documentation of the operational principles involved in common installation and sampling techniques (Johnson Division, UOP inc., 1975; U.S. EPA, 1975; Scalf and others, 1981; Barcelona and others, 1985). This subsection discusses the advantages and limitations of those techniques the author has employed at hazardous waste sites. Case histories are presented in the next section to further illustrate the problems encountered and the techniques devised to overcome them.

3.2.1 Rotary Drilling Methods

Rotary methods (Figure 3.9) have been extensively employed in the drilling of wells for resource development; e.g., water supply wells, oil and gas wells, and geothermal wells. One of the reasons for the popularity of rotary drilling methods is that they are rapid; in excess of one hundred feet of borehole advancement per day is common. The use of rotary methods for the installation of monitoring wells at hazardous

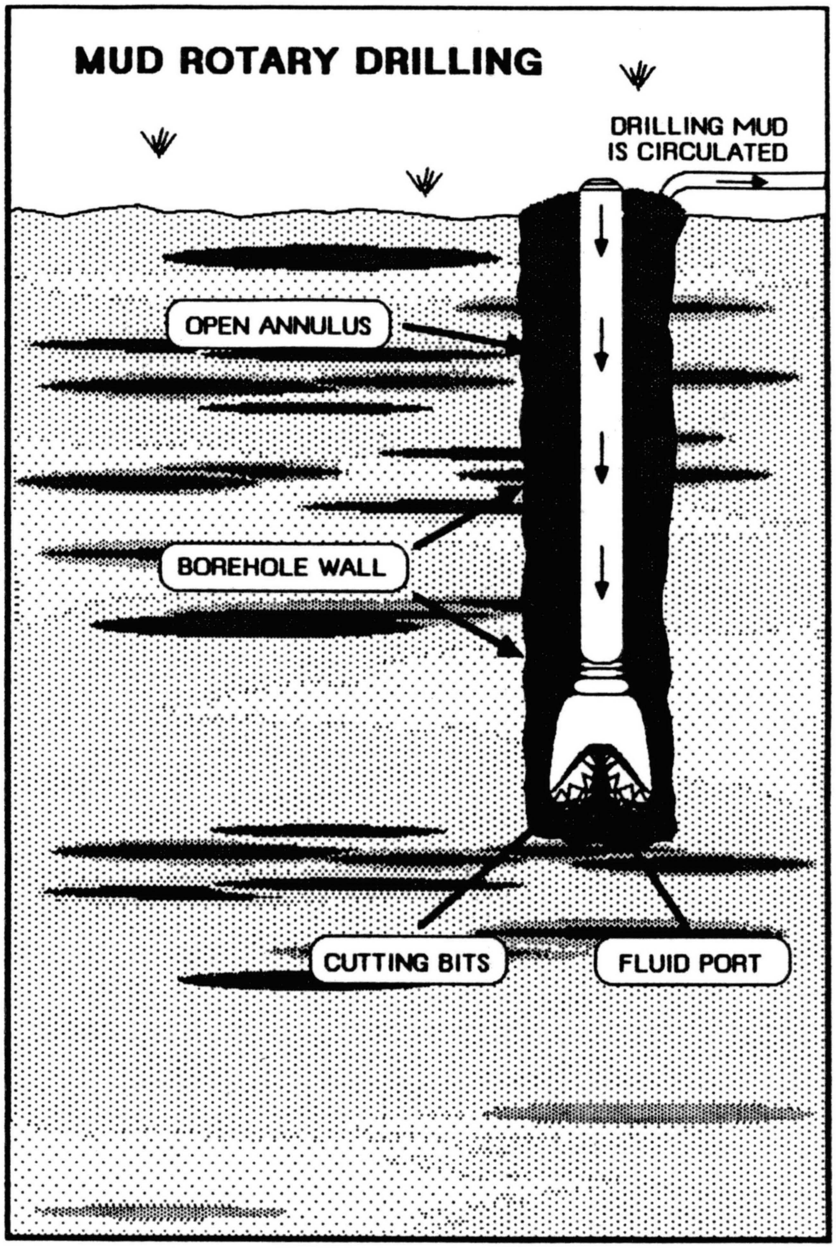


Figure 3.9. Cut-Away Sketch of Mud Rotary Drilling Method

waste sites has been generally limited to drilling in off-site areas or through formations known to be free of contamination. These limitations are the result of concerns for potential cross-contamination of strata exposed to the mud circulating in the borehole during drilling.

Even these limited uses of mud-rotary drilling at hazardous waste sites are fading quickly, however, because of uncertainties about the ability of well development efforts to remove drilling mud residues. Mud remaining in the formation after drilling may lower the permeability locally, causing certain strata to yield lesser amounts of water than they should. Mud residues may also serve to alter the ground-water chemistry by binding metals, sorbing organics, supporting excessive biological growth, and altering the cation exchange capacity (CEC), pH, and chemical oxidation demand (COD).

3.2.2 Cable-Tool Drilling

The cable-tool drilling method (Figure 3.10) has been extensively employed for the installation of water supply wells, but has been used only infrequently at hazardous waste sites. Its use may be advantageous in hazardous waste situations, especially where hydrogeologic conditions are variable or not well known. This is because the cable-tool drilling method employs the use of a temporarily emplaced solid casing that primarily serves to keep the borehole from collapsing during drilling, but that also serves to minimize possible cross-contamination between strata. At sites where the distribution of contaminants within the aquifer(s) is not well defined, a drilling method that allows sampling of subsurface solids and fluids during well installation would be

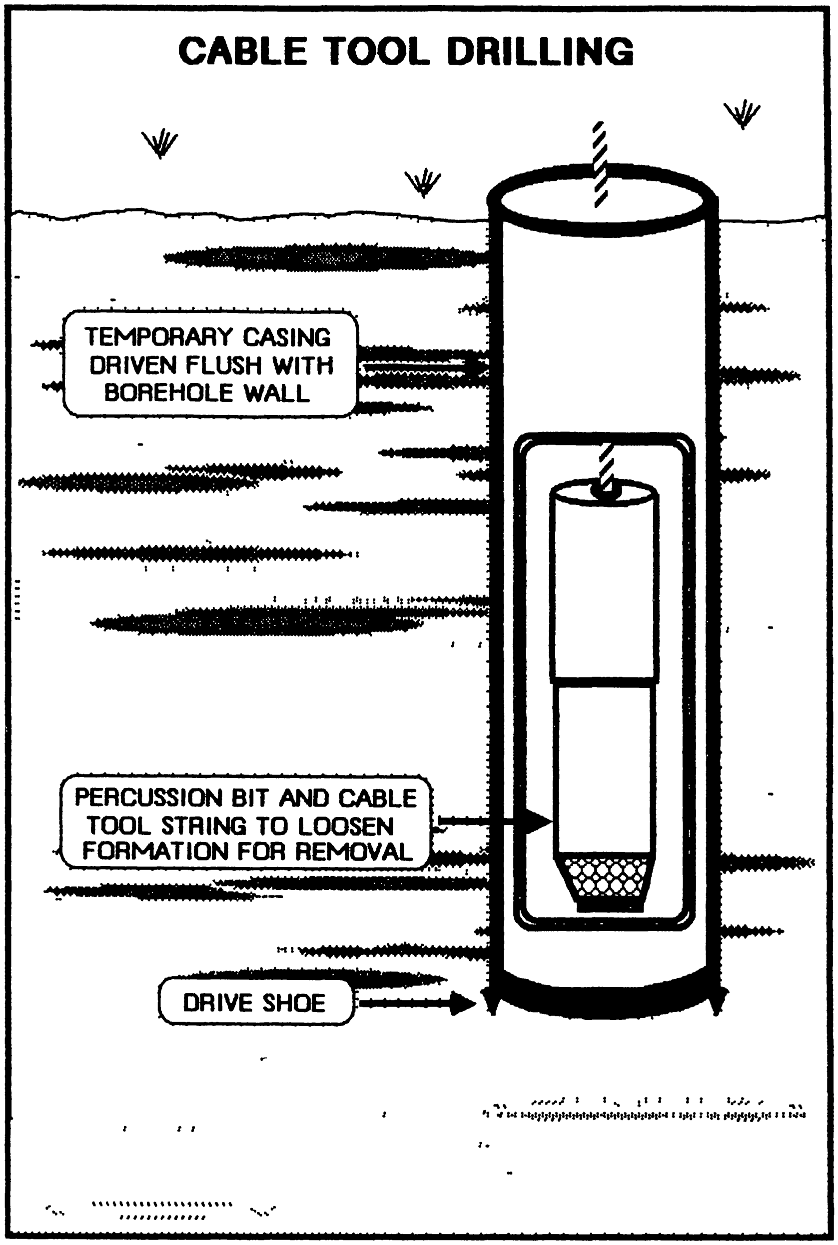


Figure 3.10. Cut-Away Sketch of Cable Tool Drilling Method

advantageous. The cable-tool method is superior in this respect; it allows relatively undisturbed samples to be collected.

In cable-tool drilling, the borehole is advanced by lifting and dropping a heavy string of drilling tools which are suspended on a steel cable and terminate in a chisel shaped bit. The impact of the bit breaks up the formation and the resulting material must be removed from the borehole. In unconsolidated formations it is necessary to drive a temporary steel casing to prevent collapse of the borehole. The temporary casing is equipped with a sharp drive shoe attached to the lower end, which aids the advancement of the casing by carving out a slightly larger diameter bore than made by the action of the bit. Water is sometimes added (usually necessary above the saturated zone) and the loosened material is mixed by the up and down motion of the drilling tools, to form a slurry for ease of removal by a bailer or a sand pump. The process of driving the casing, drilling, and bailing is repeated until the temporary casing is at the required depth.

The permanent well casing with its screen at the lower end is then lowered inside the temporary casing to the desired depth, and the process of gravel packing and grouting is carried on while withdrawing the temporary casing. The latter process takes some experience and skill because the gravel pack and grout tend to slump in place. The techniques that have been found to be most useful in avoiding such difficulties are:

1. A tri-cone bit or under-reamer may be used during drilling for those clay strata which are dry or nearly so when first encountered, so that the temporary casing will not be trapped in the expanding clay as it becomes wetted by further work in the borehole.

2. A temporary casing of sufficiently large diameter should be used so that an annulus of at least three inches surrounds the permanent well when it is placed in the temporary casing.
3. Fine gravel should be used for the packing material to minimize the potential for bridging (often a problem with pea-gravel) and to reduce the amount of well development and purging, by more effectively filtering the aquifer fines.
4. At least thirty percent more than calculations of the borehole volume indicate are minimally necessary for gravel and grout should be emplaced to allow for considerable in-place slumping.
5. The gravel pack and grout should be emplaced, and the temporary casing withdrawn, in two- to five-foot stages.

One advantage of cable-tool drilling is that it allows representative water and solid samples to be collected during drilling. The data obtained from such samples helps to determine the optimal placement of the well screen (Boateng and others, 1984). Arbitrary or 'blind' placement of well screens can result in poor definition of the contaminant distribution within the aquifer.

The annular space between the formation and the temporary casing is ordinarily negligible with cable-tool drilling, unlike augering. The amounts of water added to the borehole are usually small, and these tend to remain inside the temporary casing. Hence, there is little likelihood of the potential cross-contamination that might occur by moving contaminated fluids or solid material from one stratum to another. Driving and removing the temporary casing does not result in significant disturbance of the sediments encountered because of the smoothness of the casing and the slowness of its advancement. This can be a major advantage in fluvial and glacial sediments, which typically have interstratified silt and clay lenses that tend to smear with augering. Another useful advantage to the traditional techniques of cable-tool drilling is that a sufficiently large diameter borehole may be drilled so as to enable multiple-well completion within a single temporary casing. This is an extremely complicated task to carry out successfully, however, and is advisable only when drilling several adjacent boreholes would not be feasible economically (e.g., great depth) or technically (e.g., limited space).

The major disadvantages of the cable-tool drilling method are the time and costs involved. It takes a minimum of one field day to withdraw an 8" diameter steel casing from a depth of about 100 feet in an unconsolidated formation if hydraulic jacks are not used; where expanding clays are present, we have observed progress as poor as a few tens of feet per field day. The nominal cost typically exceeds $20 per foot of depth drilled; however, this cost must be viewed in balance with the lowered costs gained by the need for less development of the well constructed, which results from the ability to emplace a large and effective gravel pack.

3.2.3 Hollow-Stem Augering

Hollow-stem augering (Figure 3.11) is fast and relatively inexpensive. Several hundred feet of borehole advancement per day in unconsolidated sediments is possible. The cost per foot of borehole is about $10-$15. These factors alone make it competitive with rotary

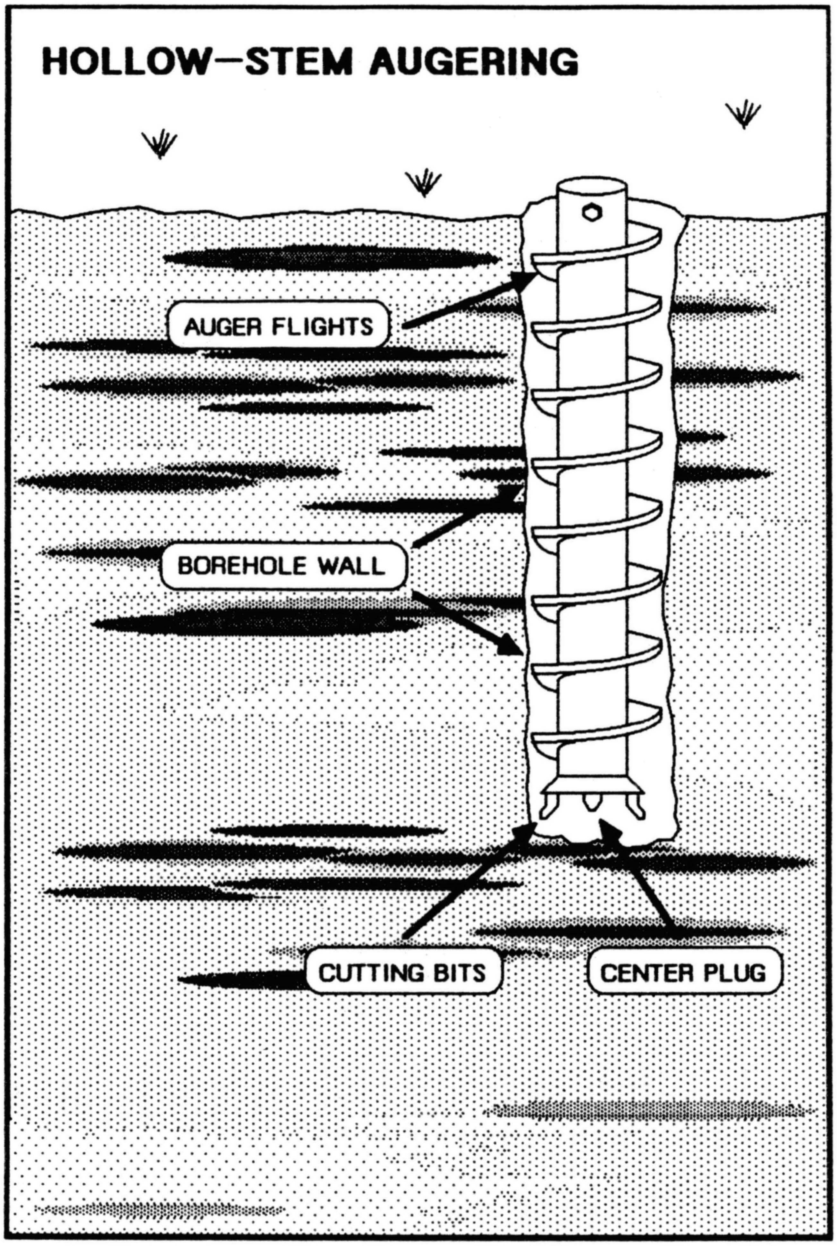


Figure 3.11. Cut-Away Sketch of Hollow-Stem Augering Method

methods. For ground-water contamination investigations it is preferred over rotary methods, however, because:

1. No drilling fluid need be used.
2. It is believed that each stratum encountered may be prevented from contacting fluids or solids from other strata (see criticisms below).
3. Solid samples are readily retreived by split-spoon samplers during the course of drilling.
4. One can remove the center plug when the target depth has been reached and immediately begin construction of the monitoring well inside the hollow auger flights.

When augering, the hole is advanced by rotating and pressing the auger into the soil. As the auger is pressed into the soil, cuttings are rotated upwards on the auger flights. This poses a potential cross-contamination problem since contaminated material from a lower stratum may be brought into contact with an uncontaminated overlying stratum. The augering action also causes interstratified clays and silts to smear into open sand and gravel strata, possibly changing local permeabilities and affecting the proportion of flow delivered from each stratum to the monitoring well.

To ensure that the auger does not become bound to subsurface materials, binding-prevention techniques are employed. These generally consist of rotating the auger flights in place, or sequentially raising and lowering the flights a few feet while they are rotated; ordinarily, this is done every few feet of borehole advancement. Such actions aggravate the smearing of clays and silts into other strata, because the vigorous action of the rotating flights pushes loosened material against the borehole wall. Moreover, binding-prevention techniques enlarge the borehole beyond the nominal diameter of the auger flights (Figure 3.11). Thus, potentially contaminated solids or fluids from an overlying stratum may be brought into contact with a lower stratum by falling down the annular space.

The major limitations of this drilling method are that it cannot be used to drill hard rock formations and is generally incapable of being used to drill deeper than a few hundred feet. Augering is also slow in coarse materials such as cobbles and boulders. One technique found to be useful in minimizing the difficulties of augering through such sediments is the use of a small diameter tri-cone roller bit (run through the center of the auger after removing the plug) to shatter the large cobbles encountered. No drilling fluid is used since the amount of this kind of drilling is minimal so that it does not result in excessive wear on the tri-cone bit.

Obtaining representative water samples from discrete strata during augering is not too difficult; though special modifications are helpful, such as the Keck Screened Method which incorporates a screened section into the shaft the auger flights are welded to (Scalf and others, 1981). One limitation deserving of mention is that many geophysical logging techniques cannot be used, as a result of interference from the thick metal walls of the auger (the same limitation applies to the cable-tool method if the temporary casing is steel, though schedule-80 PVC casing has the required strength and may be used). Finally, hollow-stem augers are particularly susceptible to 'blowback', wherein the removal of the central plug is accompanied by an immediate rise of loose sediments inside the lower auger flights. This can be quite severe (i.e., several tens of feet of sediments) and complicates the retrieval of representative solid samples.

3.2.4 A Hybrid Drilling Method

The foregoing discussions indicate that none of the conventional drilling methods is without its technical or economic disadvantages. Clearly, the rapid advancement of the borehole by augering and rotary drilling, and their relatively low costs, are desireable features of these techniques. The complete isolation of strata encountered with temporary casing, and the superior capacity for gravel pack emplacement, are key advantages of the cable-tool method. It is only reasonable to want all these advantages in a single technique.

Figure 3.12 illustrates one possibility that has been field tested by the author and found to be satisfactory. The use of temporary casing is borrowed from the cable-tool method, and borehole advancement is accomplished by either an auger or a rotary drill that is run through the center of the temporary casing. Augering is preferred over rotary drilling because no foreign fluids need be introduced. This is not strictly necessary, however, because clean water may be used with rotary drills since the drilling fluid need only be used to cool the bit; a heavy drilling mud is not required to stabilize the borehole walls, as this is taken care of by the temporary casing. In either case, the borehole is advanced a foot or two at a time, followed by driving the temporary casing to the new bottom-hole depth.

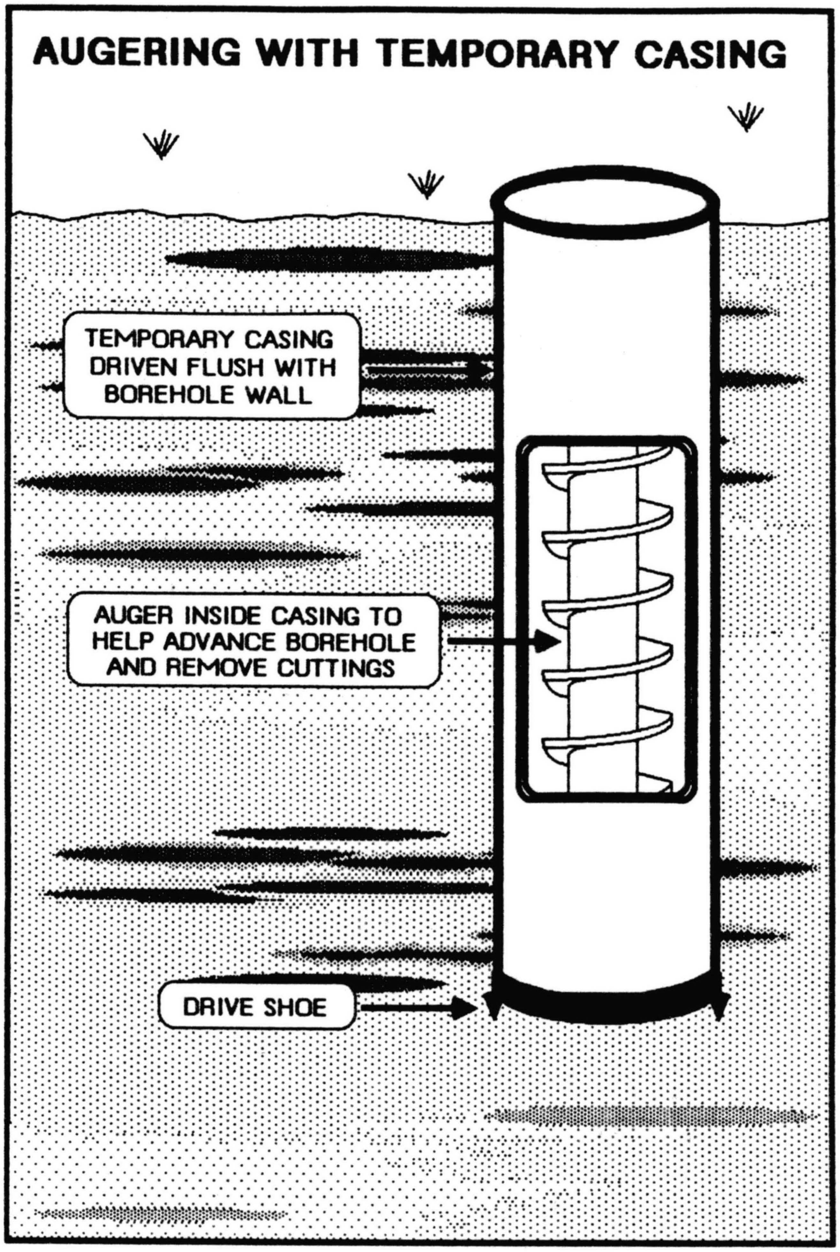


Figure 3.12. Cut-Away Sketch of a Hybrid Drilling Method -- Augering Within a Temporarily Driven Casing

Why, then, the emphasis on augering as the borehole advancement device? First, if a hollow-stem auger is used, it may facilitate the rapid acquisition of subsurface sampling with common devices like split-spoon samplers and Shelby tubes; it is usually easier to withdraw the central rod and bottom plug from a hollow-stem auger than it is to withdraw the entire drill stem on a rotary drill rig. Secondly, the use of any drilling fluid is cause for some concern, since the native fluids are displaced and diluted in the process; quite a lot of fluid may be required to keep cuttings moving up the annulus between the temporary casing and the drill rod stem. This means that obtaining samples of native fluids during the drilling process, as for screening-level determinations of which strata may be contaminated, will be difficult to impossible. It also gives rise to the need to re-think the extent of well development and purging that may be necessary when well construction has been completed and sampling is to commence.

There are some potential difficulties to consider when advancing the borehole with an auger inside a temporary casing, however. The first of these is the potential for bridging of loosened sediments between the auger and the inside of the temporary casing; in alluvial and glacial sediments that typically have widely varying grain sizes this may indeed occur. The experience gained with this hybrid drilling method thus far indicates that, while bridging has occurred, the rigs used were powerful enough to grind away and free the auger without too much problem. Helpful techniques in this regard included reversing the rotation direction of the auger, adding or releasing pressure on the flights by attempting to sequentially raise and lower the auger flights, and brutalizing the assemblage with a sledge hammer (always a favorite with 'real men').

A second major difficulty that can occur with this technique relates to the inability of hollow-stem augers to penetrate zones containing large cobbles or boulders, since there is little slippage allowed by the temporary casing. The easiest way found to overcome this problem was to remove the center rod and bottom plug from the hollow-stem auger, and run a small tri-cone roller bit down to shatter the obstructing cobbles or boulders. Little or no drilling fluid need be used for this, because the bit is downhole for such a short period of time.

This same temporary use of a tri-cone bit can be used to alleviate the other major problem so far encountered with the hybrid technique discussed here; that of potentially locking the temporary casing in formerly undersaturated, swelling clays. A small tri-cone bit (two to three inch diameter) is allowed to 'bang around' just ahead of the entire assemblage, gouging out a couple of extra inches of annular space for the temporary casing to be advanced through. Large bits may gouge too much, potentially creating undesireable pathways between the temporary casing and the borehole wall.

3.2.5 Well Development Methods

After a monitoring well has been installed, replete with gravel pack and grout, it is important to develop the well to ensure maximum removal of fines from the vicinity of the screen. Not only does this ensure free flow of fluids from the formation into the well, but it substantially minimizes the amount of free fines that may accumulate in the well between sampling events (thus reducing purging requirements). It also serves to aid in the build up of an effective filter pack, so that turbid samples are not obtained after minimal purging.

Of the methods available for well development, over-pumping is probably the most effective. A high capacity pump is lowered into the well and allowed to pump at a rate rapid enough to nearly draw the well dry. It is then switched off and the well is allowed to recover naturally; which is followed by several more cycles of evacuation and recovery. Generally, the target is to reach a point in these cycles where sediment-free water is all that is produced. Personal observations indicate that nature has a tendency to ignore this generalization, however; a second series of evacuation-recovery cycles is recommended after resting the well for twenty-four hours or so. Apparently, some settlement and further loosening of the fines occurs after the first development attempt; a third attempt is probably not justified, since the second attempt is usually successful after a much shorter period than the first.

Another popular, but less effective method, of development is jetting with air or water. In this method, a small diameter pipe is lowered into the well to a point a few feet above the well screen, and water or air is injected into the well through the pipe under high pressure. The water column in the well is thus vigorously mixed so that sediments at the bottom are geyesered out the top of the well. The well can be partially evacuated this way, but little loosening of fines just outside of the gravel pack occurs; hence additional development by recovery of the well is less effective than desired. It is important not to jet air or water directly across from the screen; either may cause fines to be driven into screen entrance openings (causing irretreivable blockages). Air may become entrained behind the screen (reducing the effective permeability locally), and jetted water will substantially displace native fluids so that excessive purging may then be required prior to sampling.

3.2.6 Purging Techniques

When sampling monitoring wells for evaluation of the quality of water in the aquifer, it is important to remove stagnant waters stored inside the well casing and waters immediately adjacent to the well that are in the formation. Metals in these waters, such as iron and manganese often precipitate when exposed to the oxygen-rich atmosphere, which may cause the co-precipitation of other metals such as arsenic, cadmium, and lead, and which may serve as sorption substrates for organic chemicals. This effect can be quite striking; the author has observed the formation of brown (iron?) oxides within minutes of obtaining fresh samples from pumping wells.

Gases in solution in the aquifer, such as carbon dioxide, will try to reach new equilibria with atmospheric gases, dramatically altering the pH of the sample and potentially stripping volatile organic chemicals from the sample. The author has observed samples that literally frothed with escaping gases as the samples were produced. Contaminants may also react with the well casing materials they are exposed to, either sorbing to the materials or leaching other contaminants from those materials (Barcelona and others, 1985). Finally, accumulated sediments in the bottom of monitoring wells would serve to distort the contaminant level determinations if they were allowed to remain during sampling. Hence, the need for purging wells is clearly established.

How much should one purge a monitoring well? That seems to have been one of the million-dollar questions of the past decade, and it is still the subject of much controversy. Five years ago, the most complete publication on sampling techniques (Scalf and others, 1981) summarized literature discussions and practical handling of the issue by noting that removal of somewhere between four and ten borehole volumes of water was common. It went on to say that some investigators argued that a borehole must be completely evacuated during purging regardless of the amount of water withdrawn, and that others recommended that stabilization of temperature and pH of withdrawn waters would be an adequate indication sufficient purging.

More recently, EPA issued a definitive study entitled Practical Guide for Ground-Water Sampling (Barcelona and others, 1985). That publication adopts a twin strategy of specifying the minimal volume to be purged by consideration of the hydraulic characteristics of the well in conjunction with stabilization of Eh, pH, temperature, and conductivity (specific conductance, $1/\Omega$) over at least two borehole volumes. The purging device was to be located in the screened interval of the well, and was also to act as the sampling device.

Despite the recommendations of this recent EPA publication, most EPA field personnel and their contractors presently do things the way they have been doing them for years; purging is accomplished by bailing or use of low-flowrate pumps, and samples are taken with stainless steel or teflon bailers. There is rarely any attempt to monitor the stabilization of Eh, pH, temperature, or conductivity. Because of this state of affairs, it is fruitful to examine the merits of prevailing and newly recommended purging criteria.

An examination of the prevailing method will show that simply specifying a minimal number of borehole volumes, without consideration of the rate of withdrawal, can lead to ineffective voiding of stagnant waters in the casing. The author alluded to this issue previously (Keely, 1982; also, see discussions in the next section of this chapter) using formula developed by Papadopoulos and Cooper (1967) that allow the relative contribution from stagnant waters to be computed; and the new EPA publication expands on those concerns (Barcelona and others, 1985). The gist of these presentations is that stagnant waters in the casings of wells tapping moderate to highly transmissive aquifers are voided rather quickly if purging proceeds from a point near the screen and at a rate sufficient to create significant drawdown on the well. Stagnant waters in the casings of wells tapping low transmissivity aquifers are removed almost instantaneously if a high-flowrate pump is utilized; otherwise they comprise the bulk of pumped waters for a moderate length of time, as the formation contributes little water due to its low transmissivity.

The question to ask in this regard is whether purging devices are frequently used that do create significant drawdown in the monitoring well; this is rarely the case. Rather, so much concern has been expressed over the materials the purging devices are made of that most field practitioners use bailers or specially constructed pumps that are incapable of pumping more than a few gallons per minute; e.g., the

Johnson-Keck SP81™ that is constructed so that ground water is exposed to stainless-steel, teflon, and high-density polyethylene contact surfaces only, and has a nominal flowrate of only one gallon per minute. In moderate to highly transmissive aquifers, where most of the Superfund sites seem to be found, such flowrates create unimpressive dimples in the water table.

Instead of decrying the situation as hopeless, however, there may be another alternative. This amounts to modifying the pumping strategy so that a purge pump is not automatically placed in or just above the screened interval, but is instead lowered to a point just below the water table prior to purging. Pumping then commences, and after purging the equivalent of one borehole volume from that position, the pump is lowered while still pumping to a point approximately one-quarter the distance from the original water table to the bottom of the well; the process is then repeated twice more, so that the final position of the purge pump is one across from the screen -- where it is allowed to remove another couple of borehole volumes (Figure 3.13). For particularly deep wells that have extensive columns of stagnant water in casing storage, it would be adviseable to incorporate a greater number of stages; say, six or eight instead of four.

What this purging strategy ensures, to the extent practical, is that the stagnant waters in casing storage have been selectively removed prior to purging from the screened interval. Then, when the usual bailer is lowered for collection of samples, it does not pass through waters that have been in storage for weeks or months on its way to the screened interval, but instead passes through waters that were in the formation

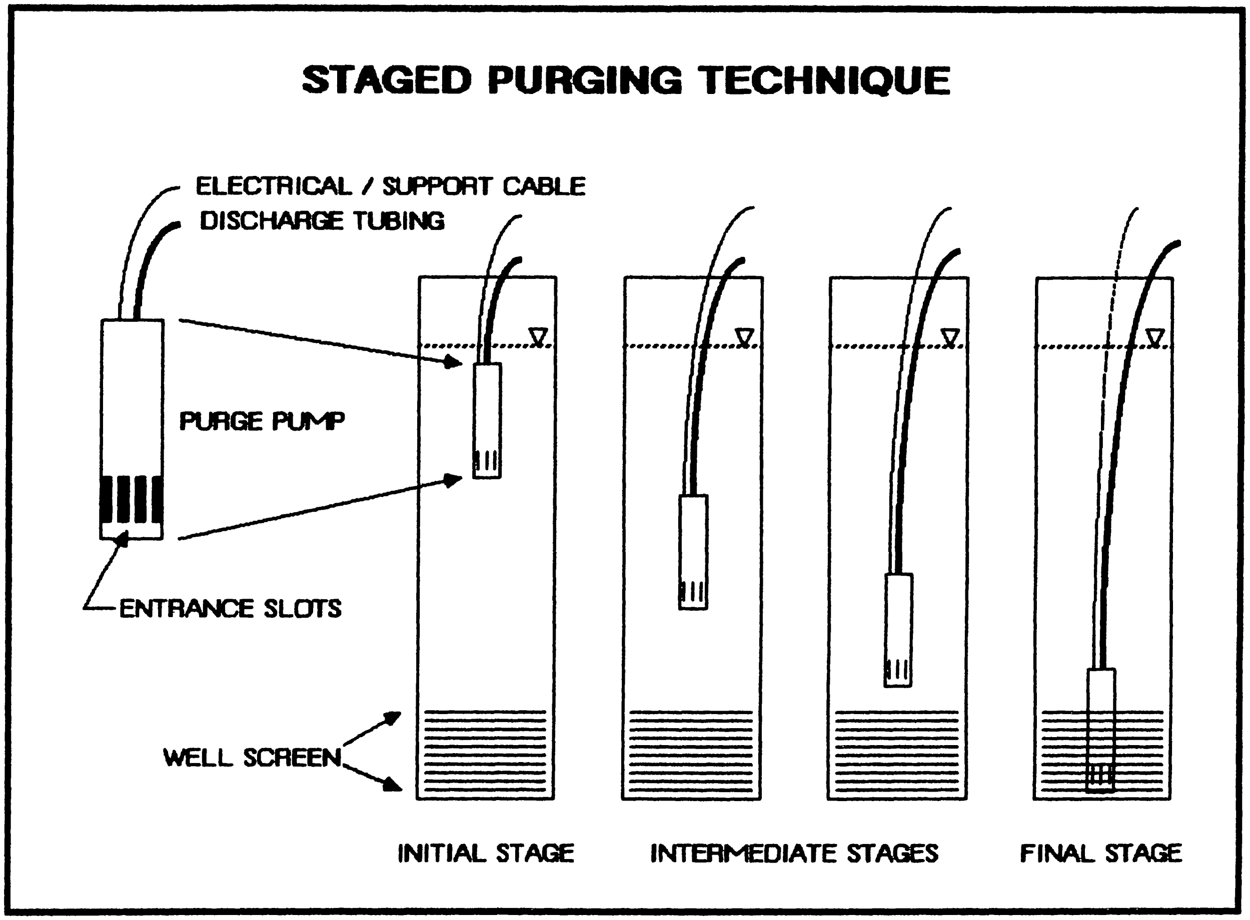


Figure 3.13. Staged Purging Technique for Ensuring Complete Removal of Stagnant Waters in Casing Storage

until only minutes beforehand. One can still object to the fact that even minutes of exposure to the atmosphere can produce undesireable changes in water chemistry as per the comments above, and that one would prefer to see the use of purging pumps that can act as sampling devices immediately after purging is accomplished and without removal of the device (such as is possible with the new bladder pumps). But those arguements have not persuaded the many doubters that cling to the use of bailers for collection of samples for volatile organic chemical analyses. Besides, the staged purging technique advocated here would add an extra measure of certainty to the use of integral purge-sampling devices, like bladder pumps, that is hard to deny.

On the issue of checking for the stabilization of Eh, pH, temperature, and conductivity to judge the adequacy of purging, it has been the author's experience that these are criteria that are vastly oversold. As it happens, those criteria serve a useful purpose when the objective is to sample constituents that are of natural origin and are ubiquitously distributed throughout the aquifer (e.g., iron, manganese, nitrate, carbon dioxide) -- because those constituents <u>can</u> reach stable levels.

But what of contaminants that are present as plumes of highly irregular composition? As has been shown by the author previously (Keely, 1982; Keely and Wolf, 1983), such contaminants have concentration levels that often vary independently of the gross physical parameters that stabilize so well after several borehole volumes are purged. Chemically, this is quite sensible. After all, it does not seem theoretically possible that the neutrally charged volatile organic chemicals can produce measureable changes in Eh, pH, temperature, or conductivity when present in the parts per billion to parts per million range.

Ah, but skeptics will point out, there was no presumption that organic chemical contaminant levels would or should stabilize. Really? It seems that such an assumption is tacitly implicit in using the stabilization of gross physical parameters as the criterion for when to take 'valid' samples. Most practitioners seem to assume that samples so taken will be representative of contaminant levels in that part of the aquifer -- at least to the extent that they are happy to prepare finely tuned contour maps of the values and to use those contour maps to support their recommendations for further actions.

It is the author's opinion that much greater discussion is warranted regarding the inherent variability of the concentrations of contaminants associated with plumes of limited extent or variable composition. To that end, the work referenced earlier concerning the need for chemical-time series sampling (esp.; Keely, 1982) should prove an adequate launching point for such dialogues. Additional amplification of these issues is given later in this section; particularly with respect to the potential use of chemical time-series sampling for estimation of the statistical variation that one might expect, and use to characterize a sampling point in lieu of conventional isolated grab-samples.

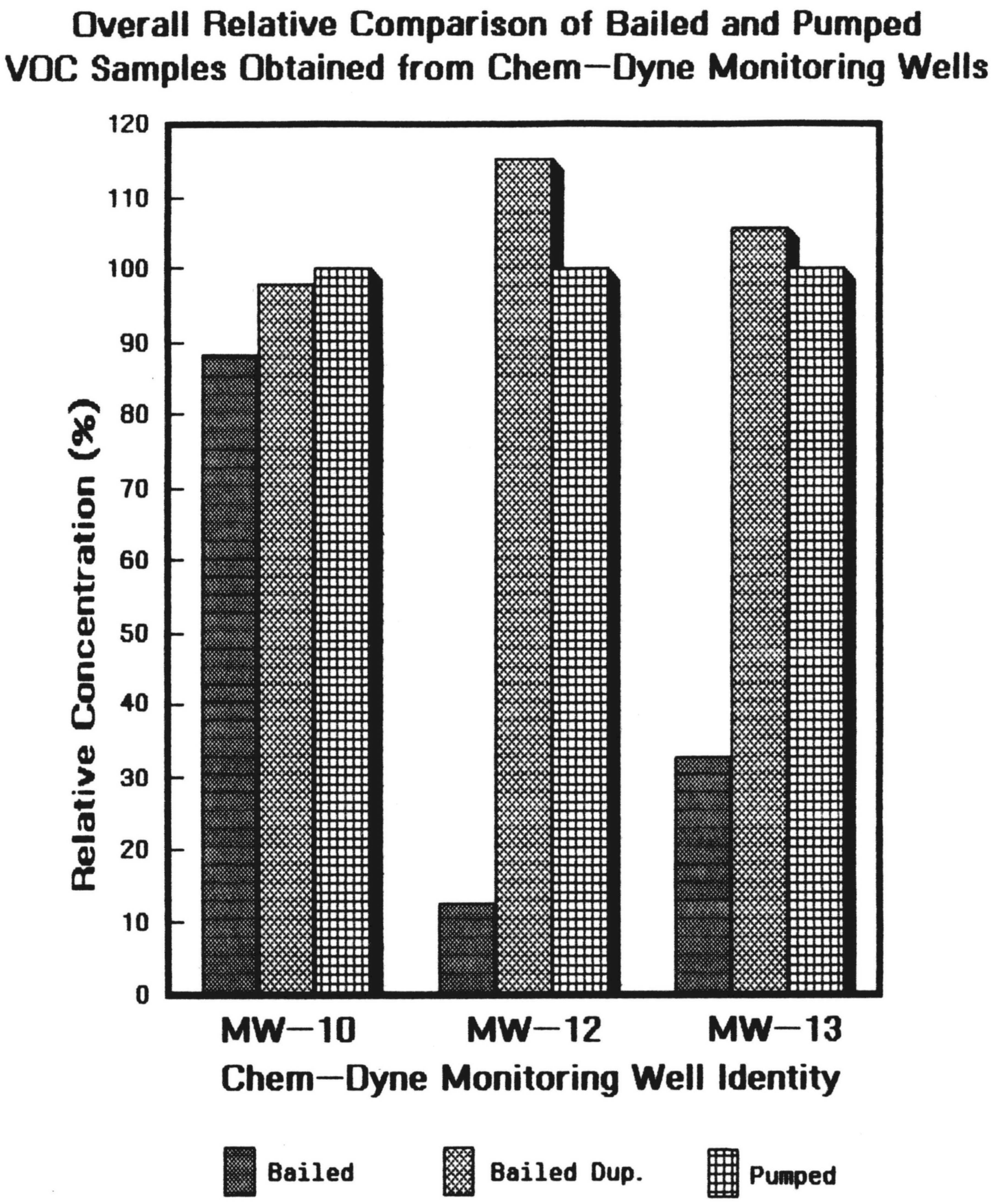
3.2.7 Sampling Techniques

In following the general theme of the previous discussion, it is apparent that concerns about the materials that sampling devices are constructed of have heavily influenced the choice of devices used. Just as importantly, concerns about the effects that sampling devices may have on contaminant concentrations as a result of their mechanical operational characteristics have driven this issue. Specifically, there has been a great deal of study of the potential for degassing or air-stripping volatile organic chemicals by sampling devices. This concern arose quite naturally, as anyone who has been in the presence of a major production well while it is producing waters laden with volatile organics can attest; enough of the contaminants are volatilized to constitute an odor and health exposure problem.

There is, however, a world of difference between the kind of pumps used for major production wells, which subject their pumped waters to severe pressure drops of relatively extended durations, and some of those pumps that are used to purge monitoring wells prior to sampling. Barcelona and others (1984) conducted an extensive comparison of purging and sampling devices that ranked bladder pumps (gas-driven positive-displacement devices that subject the sample to a minimal pressure reduction) above average in overall sampling performance, conventional bailers as average, and mechanically-driven positive-displacement type pumps (such as the Johnson-Keck SP81™ stainless-steel pump referred to earlier) as average to below average. Specifically, their study rated bladder pumps superior to bailers, and bailers superior to suction pumps, in controlling the loss of volatile organics. Unfortunately, they did not report on the performance of mechanically-driven positive-displacement pumps like the SP81™ in controlling the loss of volatiles. For the other devices, they noted increasing biases with increasing concentrations.

During the course of investigations at the Chem-Dyne Superfund site, the author conducted an ad hoc experiment of his own to determine the significance of the bias introduced by using the Johnson-Keck SP81™ to collect volatile organic chemical samples. That pump was already in use at the site for the purging of the monitoring wells and for collection of samples to be analyzed for metals, inorganic species, pesticides, and non-volatile organic chemicals on EPA's Priority Pollutant List. In the usual sampling, when purging was complete, the pump was allowed to continue discharging for collection of those samples; then the pump was quickly removed and a stainless-steel bailer lowered to collect the sample for volatile organic analyses.

On a few occasions, however, samples were taken for volatile organic analyses from the discharge of the SP81™ pump; followed immediately by sampling in duplicate with a bailer. The results of these ad hoc field comparisons are shown in Figure 3.14. For those wells that had moderate to high levels of volatiles, it is hard to distinguish the samples obtained with the SP81™ pump from those obtained with the bailer. For those wells that had very low levels of volatiles, some bias may be evident; the SP81™ samples were lower in some volatile concentrations than those reported for the bailer. These data do not, therefore, confirm trends implied by earlier published results; there is only mild indication of bias with the SP81™ and it is not persistent -- it certainly does not increase with increasing concentrations. It is the author's opinion that 'the jury is still out' on this issue, and a more thorough characteriztion of the mechanisms producing volatile losses in mechanically-driven positive-displacement devices is needed. Such



Note: Pumped samples are the reference (e.g., 100%) for the relative concentration values, based on the sum of all volatile organics detected in each sample.

Figure 3.14. Comparisons of Volatile Organic Chemical Levels Detected in Bailed and Pumped Samples

research may have consequences for the many contaminant investigations that continue to incorporate data from private and public production wells in mapping the extent of plumes. Some method of properly normalizing such data to those collected from monitoring wells with recommended devices, such as bladder pumps or bailers must be found.

3.3 Chemical Time-Series Approach

Bailers or other low volume sampling devices are commonly used to obtain samples from fairly small diameter monitoring wells for organic chemical analyses. Most ground-water professionals recognize that there may be limitations of data gathered in such a fashion; questions about 'how representative of true aquifer quality such samples are' occupy a great deal of the time and effort expended on ground-water contamination investigations. The remedies suggested to answer these questions usually fall into the areas of greater numbers and specific locations of additional monitoring wells. Generally, economic constraints are sited as the primary reason for using only a few monitoring wells to attempt to define rather complex contaminant distributions in the subsurface environment.

It is possible, however, to gain more information with fewer wells by collecting dynamic samples. That is, discharging wells can be sampled as a function of the time since pumping began (chemical time-series sampling). Each successive pumped sample represents ground water from a part of the aquifer farther from the well than the samples obtained before it. In this sense, the overall effect is equivalent to increasing the number of point-sampled monitoring wells. However, tremendous gains in efficiency and economy are also realized by sampling in this manner because hundreds of point-sampled wells would be required to sample the same volume of aquifer as is sampled by a single pumping well, after only a short period of pumping. The volume of the sample is important, because point samples are tacitly presumed to represent the average chemical quality of fairly large volumes of the aquifer from which they were withdrawn; such as occurs in numerical modeling efforts, where grid blocks of several thousand cubic feet are assigned average concentration values.

Patterns of arrival of contaminants as a function of pumpage can also be examined to yield insights into the locations of contaminant sources and plumes--much as tracer tests are used to determine preferential avenues of flow. Nearby monitoring wells that lie within the cone of pumping influence become useful for chemical time-series sampling too, analogous to observation wells for drawdown measurements during aquifer tests. Other analogies to traditional geohydraulic testing are evident. Just as the effects of natural hydraulic boundaries can be detected in such tests, artificial hydraulic boundaries are created when the cones of influence of adjacent pumping wells overlap. Contamianant sources and plumes can be effectively isolated by these boundaries so that contaminants are primarily drawn into only one pumping well and are available only to monitoring wells on the same side of the hydraulic boundary as the contaminant source or plume.

3.3.1 Mechanics of Time-Series Sampling

The mechanics of chemical time-series sampling are as follows:

1. The wells to be sampled must not have been disturbed (e.g., not pumped) for several hours to several days prior to testing. Nearby discharge wells need not be inoperative, but must remain at constant pumping rates and these rates must be known. This allows the aquifer to attain a steady-state configuration locally.
2. The static water levels of each well are measured just prior to the onset of pumping. 'Zero time' ground water samples are then taken with bailers. These samples represent relatively stagnant waters residing in the well casing--but will be important for interpretive purposes.
3. A pump in one of the wells to be tested is started and chemical samples and water level measurements are taken at logarithmically spaced intervals: e.g., 1,5,10,30,60,180 and 300 minutes since pumping started. Samples and drawdown measurements are also taken at similar times in nearby non-pumping wells. Logarithmically spaced intervals for chemical samplings are most efficient because the time required for arrival of withdrawn waters increases considerably as one gets farther and farther away from the well.
4. Just after the last samples and measurements are taken, a second pumping well is started--without shutting off the first well. The two wells will compete for waters lying within their mutually overlapping cones of depression. The artificial ground-water divide that is created separates waters drawn into the wells, and may result in isolation of the contaminant(s) on one side of the divide (Figure 3.15).

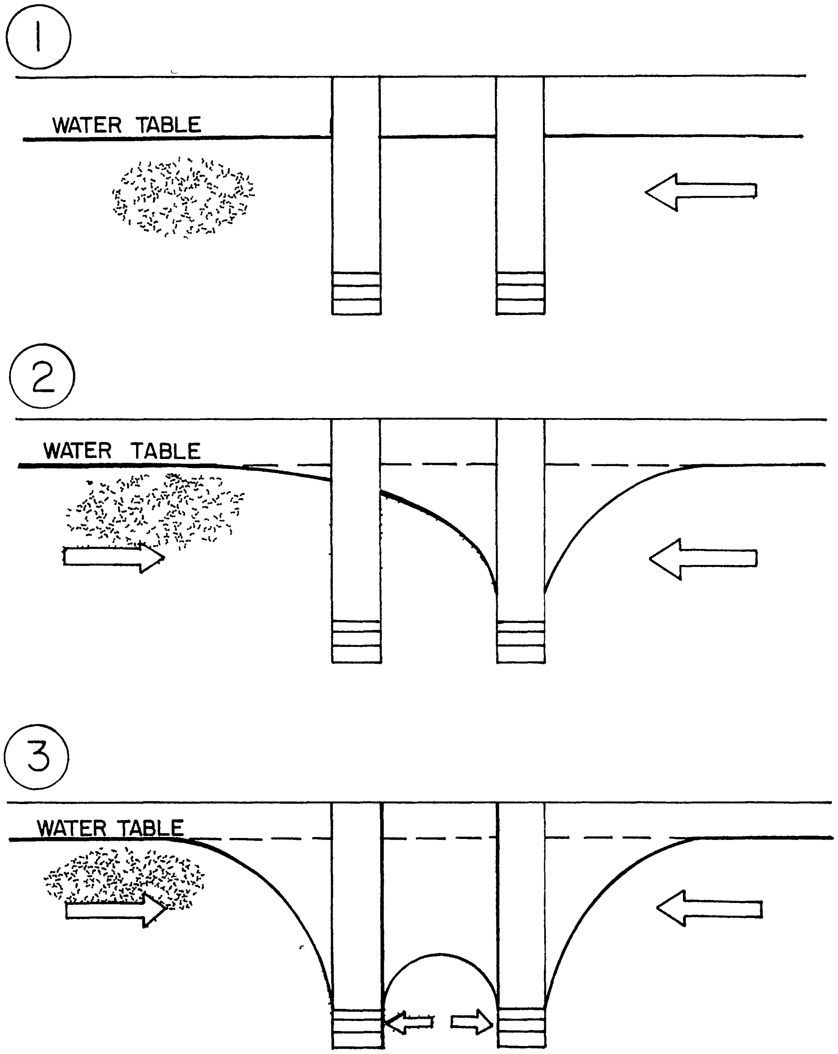


Figure 3.15. Pumping Well Contamination by a Downgradient Source, and Plume Isolation by Hydraulic Interference

3.3.2 Patterns of Contaminant Arrival

Some of the patterns of contaminant arrival which might result from time-series sampling are illustrated in Figure 3.16. No attempt will be made here to detail the litany of potential interpretations for each pattern illustrated in Figure 3.16, but some of the more likely possibilities follow.

Patterns I through IV portray variations on trends of decreasing concentrations with continued pumpage. Pattern I might result from placing the well so that it penetrates a contaminant plume of very limited extent; contaminant levels decrease due to an increase in the availability of uncontaminated aquifer waters as the distance from which the waters were drawn into the well becomes greater and greater. It might also be that the well penetrates a substantial plume, but is located close to one edge of the plume. Or it might be that a slow, but steadily increasing contribution of additional uncontaminated waters, such as from upconing or from leaky confined beds, is gradually reducing the ratio of contaminated-to-uncontaminated aquifer waters drawn into the well. Each hypothesis could be strengthened or challenged by supplemental data; e.g., if pumping tests show the existence of leaky artesian conditions locally, the last hypothosis may be most supportable. If nearby wells are unable to penetrate the same plume, the first hypothosis may be correct; and so on.

A rapid decline of contaminant concentrations after very brief periods of pumping, as shown in Pattern II, should alert investigators to the very real possibility of internal contamination of the well--such as

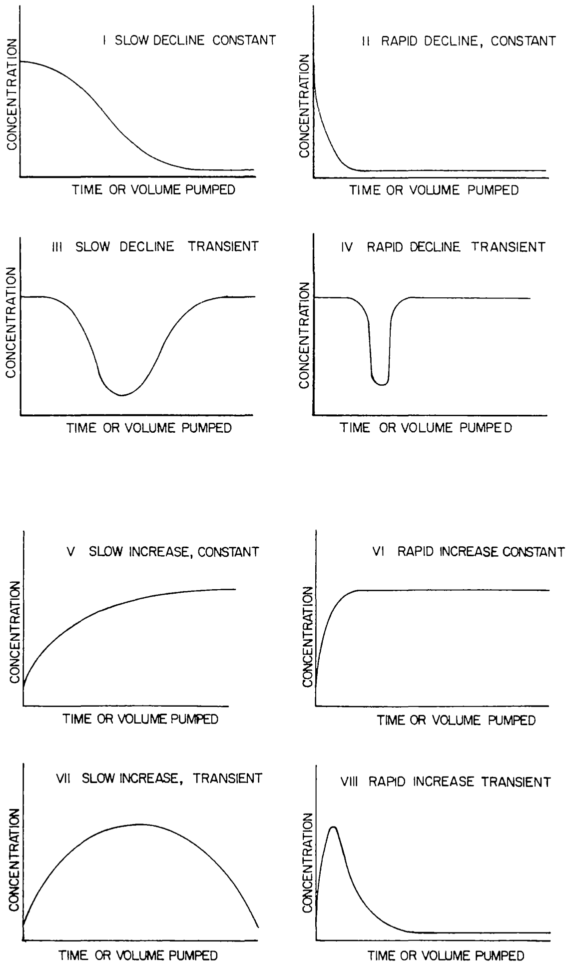


Figure 3.16. Possible Patterns of Contaminant Arrival

might occur from a poor surface seal, use of lubricating agents, or leaching of contaminants from the use of certain materials in construction of the well. As the well receives more and more uncontaminated waters from the aquifer, the concentrations of contaminants decline for each of the hypotheses just advanced. Again, supplemental data (here, regarding well construction) would be of great value in reducing the number of hypotheses that must be seriously considered.

Patterns III and IV appear to suggest not only the possibility of relatively brief encounters with recharge sources at some finite distance from a well that penetrates a contaminant plume, but also the possibility of intermittent loadings of contaminants--as might occur in fractured crystalline rock aquifers, karstic limestone aquifers and unconsolidated aquifers laced with large macropores. The more compressed the transient decline, the greater the tendency to dismiss it as a relatively unimportant anomaly. The more gradual the transient decline, the more credence that could be given to a local recharge effect. Once more, supplemental data would be of great benefit; if it can be demonstrated that a local stream provides recharge when a well is pumped, the induced recharge may well be a suitable explanation for a slow, transient decline.

Increasing trends of contaminant concentrations with continued pumpage, as illustrated by Patterns V through VIII (Figure 3.16) are also quite possible. A source of contaminants completely external to the well might explain Pattern V. The well would produce increasing concentrations of contaminants as it pulled greater volumes of the contaminant plume into it. Samples from nearby wells would be quite useful in verifying this hypothesis--both by levels of contaminants encountered, and, in the case of nearby pumping wells, by the ability to affect contaminant concentrations when creating hydraulic divides through well interference.

Pattern VI, though similar in most respects to Pattern V, would be more apt to result from physical, chemical or biological degradation of contaminants in the well itself--such as the loss of iron by precipitation from stagnant waters in casing storage between pumpage events, reported by Gibb and others (1981). The contaminant concentrations would rise rapidly as pumping continued because the stagnant, affected waters in casing storage would be quickly voided and replaced by waters from the surrounding formation.

The implications of Patterns VII and VIII are that some isolated plume is being sampled. The form of isolation may be as simple as physical occlusion by clay strata or intermittent release of the contaminants into the subsurface, or it may be a matter of relative contributions of recharge sources of uncontaminated waters versus the contaminant plume. As a final note, Pattern VIII may be the result of a combination of casing storage effects and poor well construction (leaky surface seal) or poor isolation of the aquifer of interest (communication of contaminated waters along the outside of the casing due to poor grouting). Supplemental data are, of course, essential to the interpretation of these patterns.

3.3.3 Distance of Draw Estimates

A simple approach to prediction of the arrival of a chemical contaminant from a known or selected distance to a well in response to pumpage involves the relationship of the total volume of water pumped at any time to the distance from which it came. Warner and Lehr (1977) have presented the elements of this approach for injection wells; it is extended here for pumping wells.

For example, suppose a well has been pumping for 100 minutes at a steady rate of 500 gallons per minute. The total volume of water pumped is 50,000 gallons or 6,684 cubic feet of water. Assuming that the well pulled water radially into the well, the 50,000 gallons or 6,684 cubic feet of water pumped out of the aquifer must have occupied a cylindrically shaped volume of the aquifer, centered on the well. The pumped waters were contained within the pores of the aquifer, with the remainder of the aquifer being formation materials and retained waters; so the volume of the aquifer which held 6,684 cubic feet of water must, itself, be much greater than 6,684 cubic feet (Figure 3.17). Given an estimate of the effective porosity, it is easy to calculate the aquifer volume that contained the 6,684 cubic feet of water pumped. Assuming the effective porosity of the aquifer in this example to be 0.20, the 6,684 cubic feet of water pumped were withdrawn from 33,420 cubic feet of aquifer volume.

To calculate the distance to the well from which the last few drops of water came, use the formula for the volume of a cylinder (Figure 3.17):

$$\mathbf{Vol_c = \pi * r^2 * h}$$

where: $\mathbf{Vol_c}$ is the volume of the cylinder,

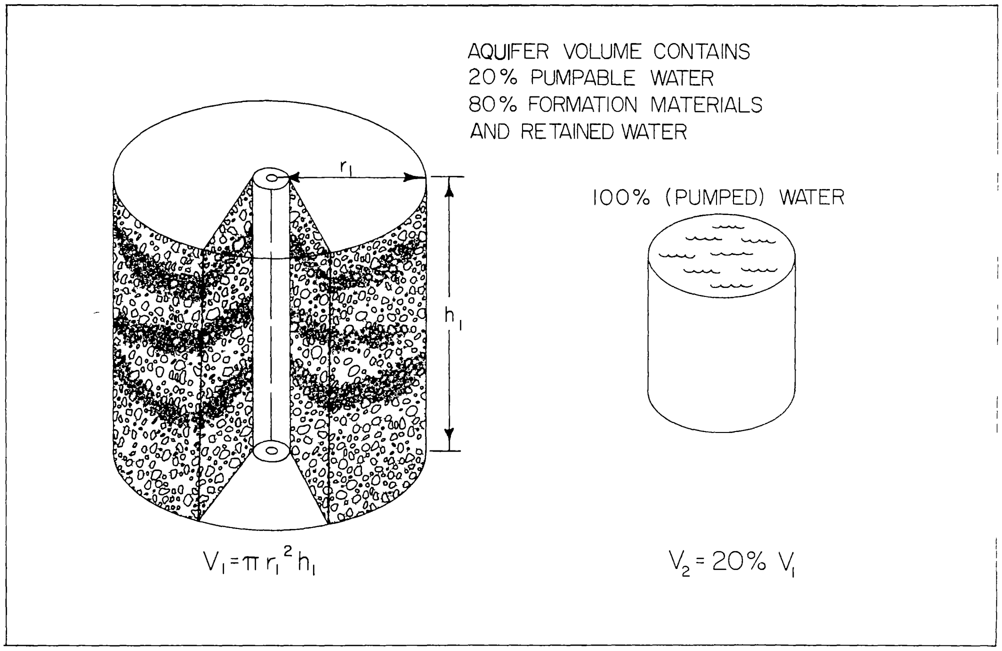


Figure 3.17. Conceptualization of Aquifer Volume that Held Pumped Waters, Assuming Radial Flow to the Well

r is the radial distance from the central axis of the cylinder to the outer surface of the cylinder, and

h is the height of the cylinder, or the effective thickness of the aquifer.

To solve for **r**, $\mathbf{Vol_c}$ and **h** must be known. $\mathbf{Vol_c}$ = 33,420 cubic feet of aquifer material in this example; and **h** must be estimated from drillers' records or other pertinent information regarding the effective thickness of the aquifer. Assume the effective thickness of the aquifer in this example to be 50 feet. In making estimates of the effective thickness, recognize that several low permeability strata may occupy portions of saturated formations that wells actually draw on; hence, it is important to distinguish between the 'effective' aquifer thickness and the nominal thickness of the zone(s) of saturation available to the well. Solving for **r**, where $\mathbf{Vol_c}$ = 33,420 cubic feet, and **h** = 50 feet: **r** = 14.59 feet.

Hence, given ideal radial flow conditions to the well during pumping, and the assumptions invoked here regarding effective porosity and aquifer thickness, the last few drops of water produced after 100 minutes of pumping at 500 gpm were drawn into the well from a distance of only about 15 feet. With the same assumptions, non-reactive chemical contaminants in those waters would have come from that distance, too. The distances of draw for contaminants that are retarded by chemical interactions can be estimated by dividing the preceeding estimate by the appropriate retardation factor; for retardation equal to 1.5, the distance of draw would be 14.59/1.5, or approximately ten feet.

Table 3.3. Radial Distance of Draw Estimates for a Well Pumping 500 Gallons Per Minute

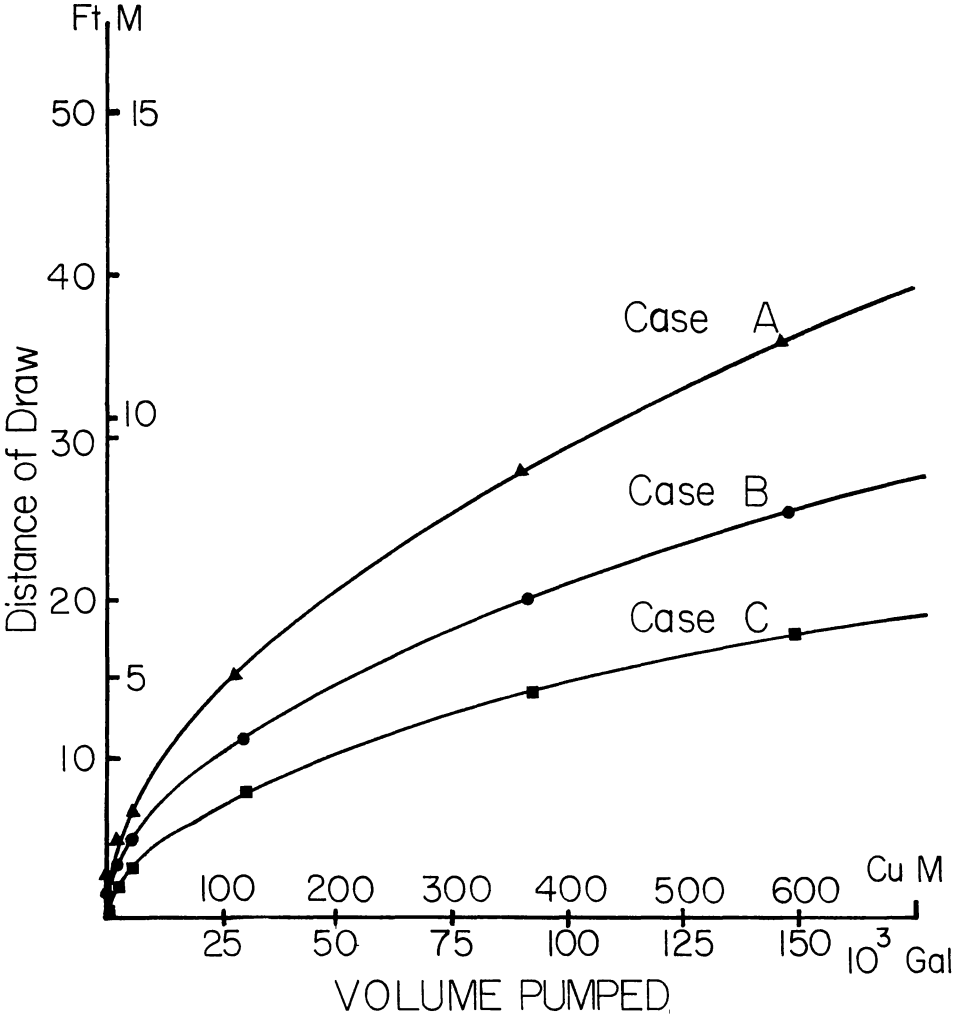
Duration of Pumpage (minutes)	Volume Pumped (gallons)	Radial Distance of Draw (feet)		
		Case A	Case B	Case C
1	500	2.06	1.46	1.03
5	2,500	4.61	3.26	2.31
10	5,000	6.52	4.61	3.26
30	15,000	11.30	7.99	5.65
60	30,000	15.98	11.30	7.99
180	90,000	27.68	19.57	13.84
300	150,000	35.73	25.27	17.87
600	300,000	50.53	35.37	25.27
1,000	500,000	65.23	46.13	32.62
1,440	720,000	78.28	55.35	39.14
2,880	1,440,000	110.71	78.28	55.35
4,320	2,160,000	135.59	95.87	67.79
5,760	2,880,000	156.56	110.71	78.28
7,200	3,600,000	175.04	123.27	87.52
8,640	4,320,000	191.75	135.59	95.87
10,080	5,040,000	207.11	146.45	103.56
11,520	5,760,000	221.41	156.56	110.71
12,960	6,480,000	234.84	166.06	117.42
14,400	7,200,000	247.55	175.04	123.77

Notes: In case A, $h = 25$ ft. and $\phi_e = 20\%$ (or $h = 50$ and $\phi_e = 10\%$).
In case B, $h = 50$ ft. and $\phi_e = 20\%$ (text discussion example).
In case C, $h = 100$ ft. and $\phi_e = 20\%$ (or $h = 50$ and $\phi_e = 40\%$).
These radial distance of draw calculations assume that the pumped waters were withdrawn from a cylindrically shaped volume of aquifer material that is axially centered on the wellbore (see text discussion).

Obviously, the power of this approach lies in its inherent simplicity. The investigator is able to make a rough correlation of the arrival time of contaminants at a well with the distance from which they were drawn. Table 3.3 contains estimates of radial distances of draw into a hypothetical well pumping at a constant rate of 500 gallons per minute from aquifers of various effective thicknesses and porosities, for specific durations. Figure 3.18 is a graphical representation of those same hypothetical data.

The bottom line of comparing the effects of various assumptions is that if either the effective porosity estimate or the effective aquifer thickness estimate is assumed to be smaller than the example just presented, then the calculated radial distance of draw is greater than in that example. Conversely, if either the effective porosity estimate or the effective aquifer thickness estimate is assumed to be greater, then the calculated radial distance is smaller. These relationships are quite sensible since, for example, a 'tighter' aquifer (lower effective porosity) would need a greater total volume to contain the same pumped waters; which,in turn, would lead to a greater calculated radial distance of draw. Note also, that a large pumping rate from an aquifer with an effective porosity of 0.002 will result in the same distance of draw estimates as a 500 gallon per minute pumping rate from an aquifer with an effective porosity of 0.20 (e.g., Table 3.3).

It is useful to examine how this approach can be adapted to solve for the distance from which a contaminant came when aquifer anisotropy and nonhomogeneity must be taken into account. Aquifers that are comprised of several strata of differing permeabilities invariably suffer from



Notes: Same as notes for Table 3.3.

Figure 3.18. Radial Distances of Draw Versus Volume Pumped from a Well Discharging 500 Gallons/Minute

anisotropy and nonhomogeneity. For example, the tremendous differences in effective porosity and hydraulic conductivity that result from depositional stratification of stream and glacial gravel and tight clay, till and silt, give rise to the widely known nonhomogeneity and anisotropic flow of glacial aquifers (Williams and Farvolden, 1969; Hendry, 1982). If waters cannot easily flow through a layer, then they will flow around, over or under that layer. Special precautions must therefore be taken to interpret flow directions based on water level measurements in such aquifers; the flow directions will not be perpendicular to water level contours (Fetter, 1981; Barker and Herbert, 1982). Chemical contaminants may thus be carried through a rather tortuous path; sometimes never reaching isolated pockets of an aquifer, sometimes traveling farther than expected.

Distance of draw calculations can be adapted to various situations involving aquifer anisotropy and nonhomogeneity by changing the assumptions about the geometry of the aquifer volume that held the pumped waters. For example, an aquifer zone that has exhibited a marked degree of anisotropy and nonhomogeneity in a single predominant direction can be assumed to limit the source of waters drawn into the well to a rectangular aquifer volume oriented along that predominant direction, rather than to a cylindrical aquifer volume.

Calculations for a rectangular case which assumes that a well draws water from a portion of the aquifer that closely resembles a rectangular box whose length is twice as great as its width are presented in Table 3.4. The formula for the aquifer volume is given by:

$$\mathbf{Vol_r = 2 * w^2 * h}$$

where: $\mathbf{Vol_r}$ is again the volume of aquifer material that contained the pumped waters,

w is the width of the rectangular box, and **2w** is equal to the length of the box, and

h is again the effective aquifer thickness.

It is easy to solve for **w**, the box width, and just as easy to multiply by two for the box length to obtain the surface dimensions of the aquifer volume that held the pumped waters. If it is assumed that the well is perfectly centered in this surface area, then the last few drops of water pumped from the well could have come from as far away as one-half of the length of the box (ignoring diagonals) or from as near as one-half of the width of the box (Figure 3.19).

Table 3.4 is a tabulation of such maximum and minimum distances for a well pumping at 500 gallons per minute. Figure 3.20 is a graphical representation of the data in Table 3.4. Table 3.4 and Figure 3.20 also include additional data and curves which illustrate the effects of reducing and increasing the effective porosity or effective aquifer thickness estimates.

A more sophisticated approach to estimating distances of draw for anisotropic situations is to adopt an elliptical geometry. The volumetric formula in this case is:

$$Vol_e = \pi * r_1 * r_2 * h$$

where r_1 and r_2 are the radii of the ellipse. Table 3.5 is a tabulation of the maximum and minimum distances for a well pumping at 500 gallons per minute in an elliptical pattern having one radius of the ellipse equal to twice the length of the other ($r_2 = 2 * r_1$). Figure 3.21 is a graphical representation of the data in Table 3.5. Additional data and curves are

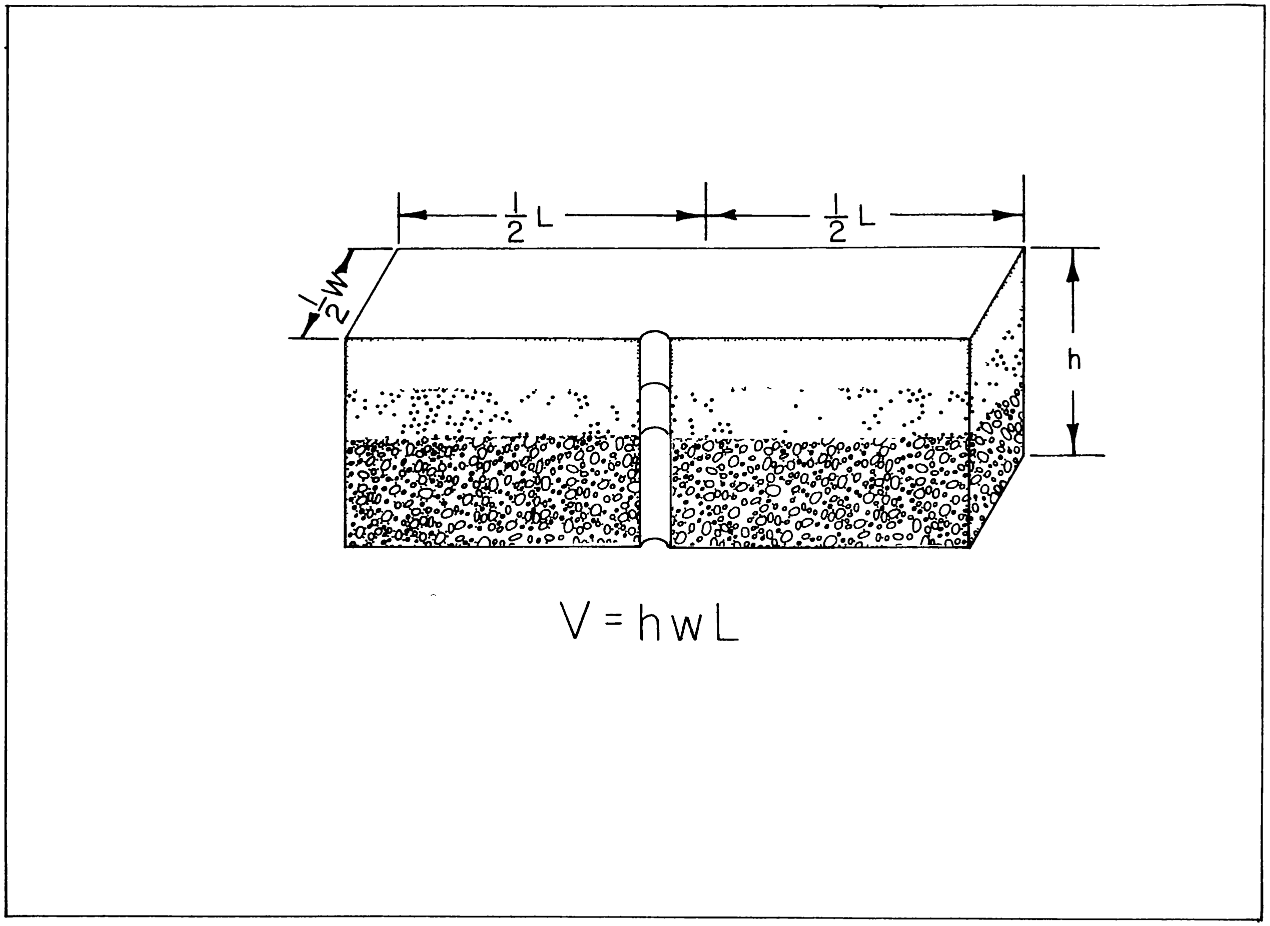
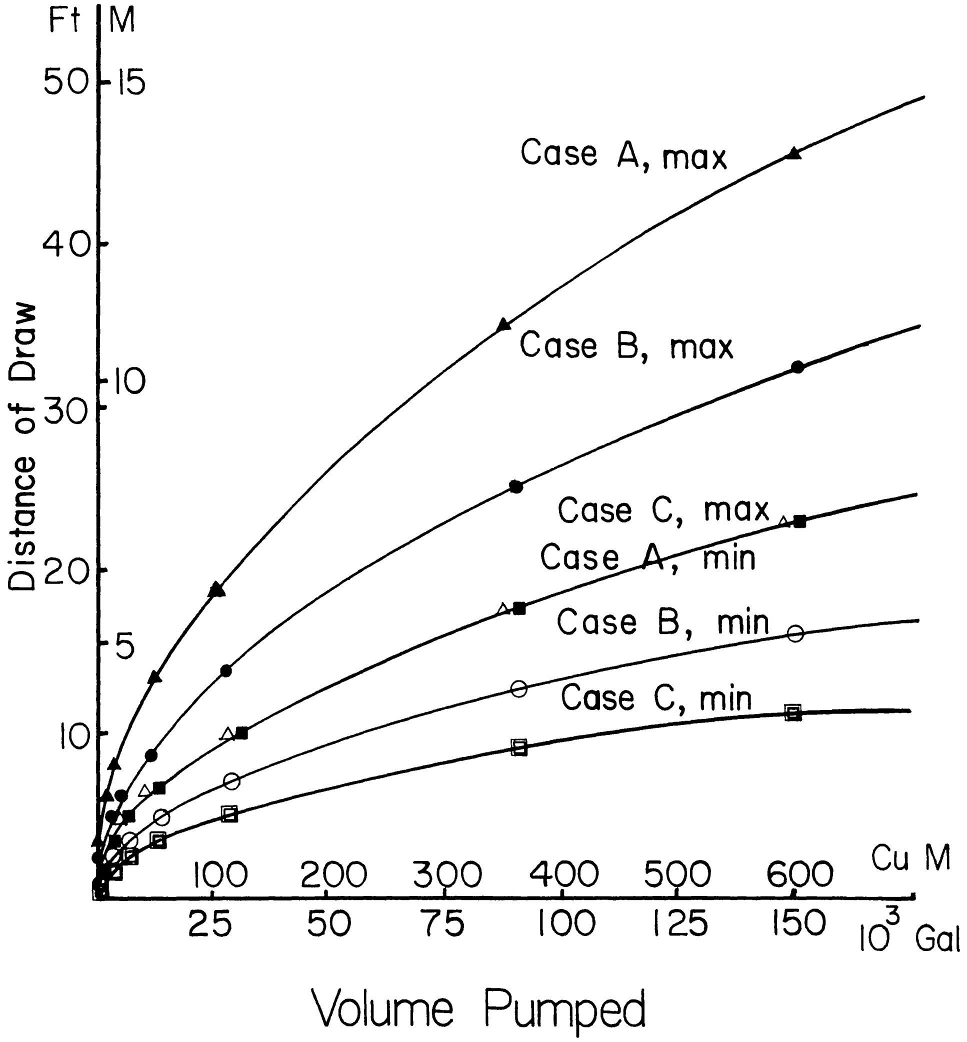


Figure 3.19. Conceptualization of Aquifer Volume that Held Pumped Waters, Assuming 2:1 Rectangular Flow Geometry

Table 3.4. Rectangular Distance of Draw Estimates for a Well Pumping 500 Gallons Per Minute

Pumpage Duration (minutes)	Volume Pumped (gallons)	Maximum : Minimum Distances of Draw (feet)		
		Case A	Case B	Case C
1	500	2.59 : 1.29	1.83 : 0.91	1.29 : 0.65
5	2,500	5.78 : 4.89	4.09 : 2.04	2.89 : 1.45
10	5,000	8.18 : 4.09	5.78 : 2.89	4.09 : 2.04
30	15,000	14.16 : 7.08	10.01 : 5.01	7.08 : 3.54
60	30,000	20.03 : 10.01	14.16 : 7.08	10.01 : 5.01
180	90,000	34.69 : 17.34	24.53 : 12.26	17.34 : 8.67
300	150,000	44.78 : 22.39	31.67 : 15.83	22.39 : 11.20
600	300,000	63.33 : 31.67	44.78 : 22.39	31.67 : 15.83
1,000	500,000	81.76 : 40.88	57.81 : 28.91	40.88 : 20.44
1,440	720,000	98.11 : 49.06	69.37 : 34.69	49.06 : 24.53
2,880	1,440,000	138.75 : 69.37	98.11 : 49.06	69.37 : 34.69
4,320	2,160,000	169.93 : 84.97	120.16 : 60.08	84.97 : 42.48
5,760	2,880,000	196.22 : 98.11	138.75 : 69.37	98.11 : 49.06
7,200	3,600,000	219.38 : 109.69	155.13 : 77.56	109.69 : 54.85
8,640	4,320,000	240.32 : 120.16	169.93 : 84.97	120.16 : 60.08
10,080	5,040,000	259.58 : 129.79	183.55 : 91.77	129.79 : 64.89
11,520	5,760,000	277.50 : 138.75	196.22 : 98.11	138.75 : 69.37
12,960	6,480,000	294.33 : 147.17	208.12 : 104.06	147.17 : 73.58
14,400	7,200,000	310.25 : 155.13	219.38 : 109.69	155.13 : 77.56

Notes: In case A, $h = 25$ ft. and $Ø_e = 20\%$ (or $h = 50$ and $Ø_e = 10\%$).
In case B, $h = 50$ ft. and $Ø_e = 20\%$ (text discussion example).
In case C, $h = 100$ ft. and $Ø_e = 20\%$ (or $h = 50$ and $Ø_e = 40\%$).
These 2:1 rectangular distance of draw calculations assume that the pumped waters were withdrawn from a box-shaped volume of aquifer material that is horizontally centered on the wellbore (see text discussion).



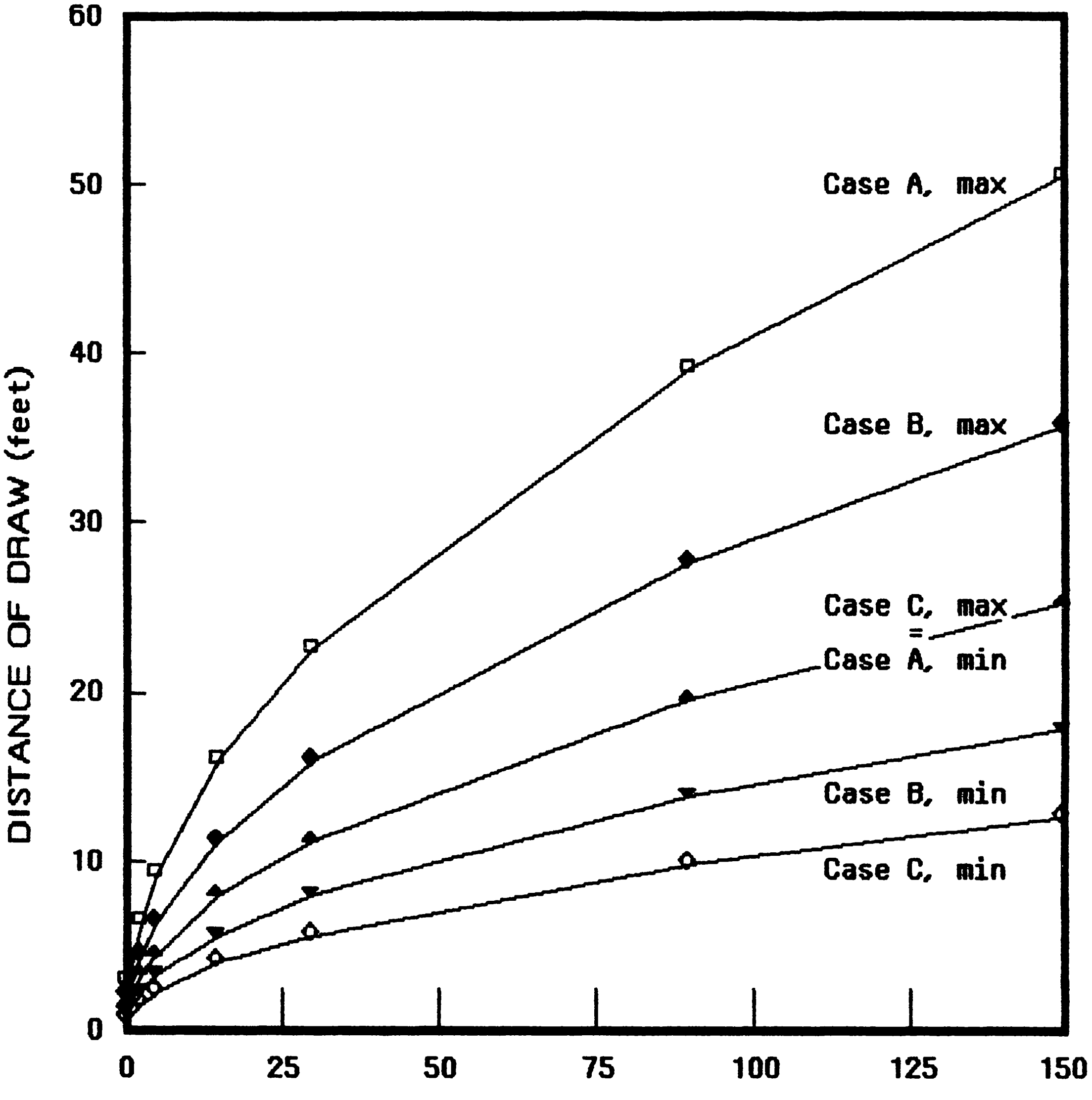
Notes: Same as notes for Table 3.4.

Figure 3.20. 2:1 Rectangular Distances of Draw Versus Volume Pumped from a Well Discharging 500 Gallons/Minute

Table 3.5. Elliptical Distance of Draw Estimates for a Well Pumping 500 Gallons Per Minute

Pumpage Duration (minutes)	Volume Pumped (gallons)	Maximum : Minimum Distances of Draw (feet)		
		Case A	Case B	Case C
1	500	2.92 : 1.46	2.06 : 1.03	1.46 : 0.73
5	2,500	6.52 : 3.26	4.61 : 2.31	3.26 : 1.63
10	5,000	9.23 : 4.61	6.52 : 3.26	4.61 : 2.31
30	15,000	15.98 : 7.99	11.30 : 5.65	7.99 : 3.99
60	30,000	22.60 : 11.30	15.98 : 7.99	11.30 : 5.65
180	90,000	39.14 : 19.57	27.68 : 13.84	19.57 : 9.79
300	150,000	50.53 : 25.27	35.73 : 17.87	25.27 : 12.63
600	300,000	71.46 : 35.73	50.53 : 25.27	35.73 : 17.87
1,000	500,000	92.25 : 46.13	65.23 : 32.62	46.13 : 23.06
1,440	720,000	110.71 : 55.35	78.28 : 39.14	55.35 : 27.68
2,880	1,440,000	156.56 : 78.28	110.71 : 55.35	78.28 : 39.14
4,320	2,160,000	191.75 : 95.87	135.59 : 69.79	95.87 : 47.94
5,760	2,880,000	221.41 : 110.71	156.56 : 78.28	110.71 : 55.35
7,200	3,600,000	247.55 : 123.77	175.04 : 87.52	123.77 : 61.89
8,640	4,320,000	271.17 : 135.59	191.75 : 95.87	135.59 : 67.79
10,080	5,040,000	292.90 : 146.45	207.11 : 103.56	146.45 : 73.23
11,520	5,760,000	313.12 : 156.56	221.41 : 110.71	156.56 : 78.28
12,960	6,480,000	332.12 : 166.06	234.84 : 117.42	166.06 : 83.03
14,400	7,200,000	350.08 : 175.04	247.55 : 123.77	175.04 : 87.52

Notes: In case A, $h = 25$ ft. and $Ø_e = 20\%$ (or $h = 50$ and $Ø_e = 10\%$).
In case B, $h = 50$ ft. and $Ø_e = 20\%$ (text discussion example).
In case C, $h = 100$ ft. and $Ø_e = 20\%$ (or $h = 50$ and $Ø_e = 40\%$).
These 2:1 elliptical distance of draw calculations assume that the pumped waters were withdrawn from an elliptical wafer-shaped volume of aquifer material that is horizontally centered on the wellbore (see text discussion). Comparison of these values with those for the rectangular assumption case (Table 3.4) will show that the elliptical distance of draw estimates are a constant 12.8% greater; hence, the error in assuming a rectangular geometry is not serious.



Notes: Same notes as for Table 3.5.

Figure 3.21. 2:1 Elliptical Distances of Draw Versus Volume Pumped from a Well Discharging 500 Gallons/Minute

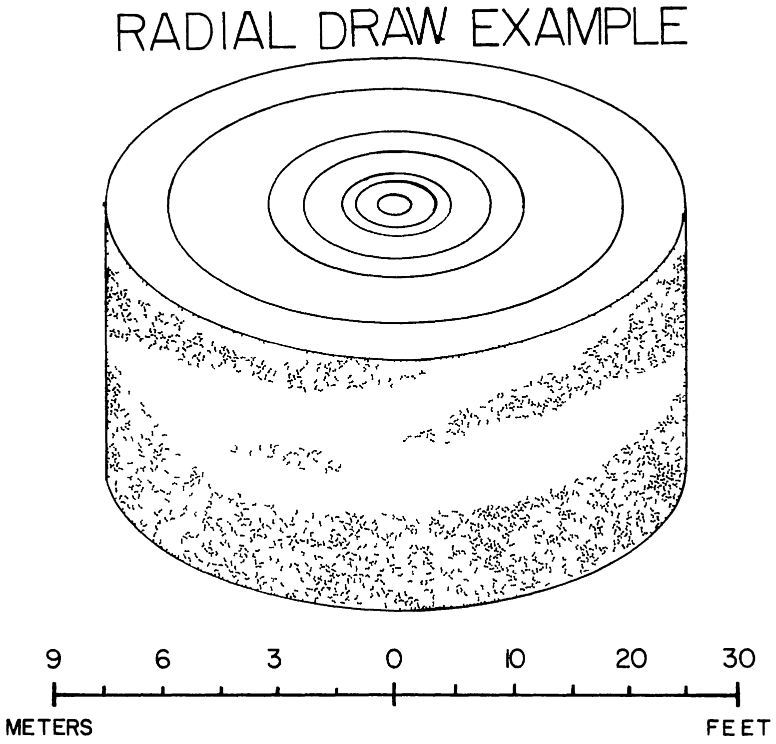
again included to illustrate the effects of reducing and increasing the effective porosity or effective aquifer thickness estimates.

It is also not unreasonable to take a hybrid approach to estimating distances of draw, wherein the initial pumping intervals are assumed to fit the ideal radial well draw approach and later pumping intervals are assumed to fit a rectangular or elliptical draw approach. Such hybrid approaches would be particularly applicable in situations where hydrologic flow boundaries occur at some known distance from the well.

3.3.4 Statistical Considerations

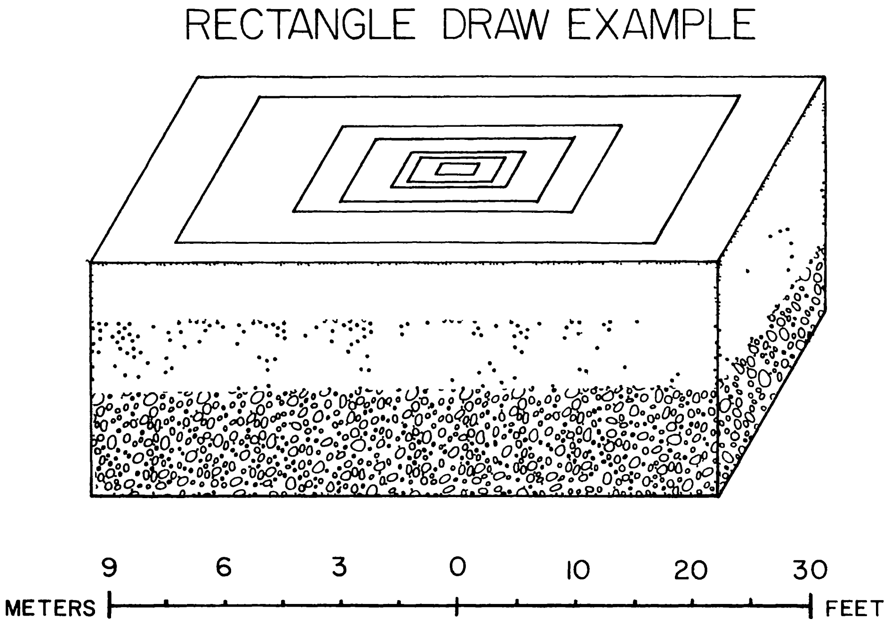
Figures 3.23 and 3.23 illustrate the aquifer zones bounded by the various radial and rectangular distances for the data given in Tables 3.3 and 3.4, respectively. Note that a conventional bailed sample would be equivalent to a zero minutes sample; i.e., it would represent an area no greater than the nominal size of the well casing. A pumped sample has a much greater chance of providing detailed information on the aquifer water quality, as opposed to a static sample.

The surface area represented by a 2-inch diameter monitoring well is equal to $\pi * r^2 = 3.4$ square inches or 0.0218 square feet. Three monitoring wells per acre (a fairly high density in conventional practice) thus nominally represent 0.0654 square feet per acre. This amounts to less than 0.0002 percent of the surface area of that acre. With adherence to the recommended purging of stagnant waters from the monitoring wells, it is likely that each monitoring well could represent as much as ten times its nominal surface area; but 0.002 percent of an acre still constitutes a statistically insignificant representation. To



Notes: Same as notes for Table 3.3.

Figure 3.22. Aquifer Zones Bounded by Radial Distances of Draw for Select Durations of Constant Pumpage



Notes: Same as notes for Table 3.4.

Figure 3.23. Aquifer Zones Bounded by 2:1 Rectangular Distances of Draw for Select Durations of Constant Pumpage

compound the problem, considerable aquifer anisotropy and heterogeneity can be reasonably assumed so that placement of the monitoring wells to ensure interception of contaminants is extremely difficult. This is why adequate representation of aquifer quality, and adequate physical descriptions based on drilling logs from monitoring wells, are so difficult to achieve.

Now assume that one pumping well and two monitoring/observation wells are available per acre. Pumping tests will considerably strengthen attempts to physically describe the aquifer, and pumped samples will represent greater volumes (and surface areas) of the aquifer. Further assume that the aquifer has an effective porosoity of 0.20 and an effective thickness of 50 feet, and that the pumping well discharges 500 gallons per minute for 100 minutes, as per the example used in the distance of draw discussions.

Again, the 50,000 gallons pumped were drained from 33,422 cubic feet of the aquifer. Dividing by the effective thickness, this means that the average composition of those drained waters represents 668 square feet of the aquifer surface area. Now the percentage of an acre of aquifer surface area represented is 1.53 percent--far more statistically significant than the 0.0002 percent contributed by each monitoring well. At 300 minutes of pumping, the percentage rises to 4.60--a very respectable value. Table 3.6 gives the results for several additional calculations of this kind so that direct comparison of the distance of draw estimates in Tables 3.3 through 3.5 may be made with the aquifer surface areas (and percentages of an acre) represented by them.

Table 3.6. Aquifer Surface Areas Represented by Samples from a Well Pumping 500 Gallons Per Minute

Pumpage Duration (min.)	Volume Pumped (10^3 gal.)	Aquifer Surface Areas (square feet) : Equivalent Percentages of One Acre (%)		
		Case A	Case B	Case C
1	0.5	13.37 : 0.03	6.68 : 0.02	3.34 : 0.01
5	2.5	66.84 : 0.15	33.42 : 0.08	16.71 : 0.04
10	5	133.69 : 0.31	66.84 : 0.15	33.42 : 0.08
30	15	401.07 : 0.92	200.53 : 0.46	100.27 : 0.23
60	30	802.14 : 1.84	401.07 : 0.92	200.53 : 0.46
180	90	2,406 : 5.52	1,203 : 2.76	601.60 : 1.38
300	150	4,011 : 9.21	2,005 : 4.60	1,002 : 2.30
600	300	8,021 : 18.41	4,010 : 9.21	2,005 : 4.60
1,000	500	13,369 : 46.13	6,684 : 15.35	3,342 : 7.67
1,440	720	19,251 : 44.19	9,626 : 22.10	4,813 : 11.05
2,880	1,440	38,503 : 88.39	19,251 : 44.19	9,626 : 22.10
4,320	2,160	57,754 : 132.58	28,877 : 66.29	14,439 : 33.15
5,760	2,880	77,005 : 176.78	38,503 : 88.39	19,251 : 44.19
7,200	3,600	96,257 : 220.97	48,128 : 110.49	24,064 : 55.24
8,640	4,320	115,508 : 265.17	57,754 : 132.58	28,877 : 66.29
10,080	5,040	134,759 : 309.36	67,380 : 154.68	33,690 : 77.34
11,520	5,760	154,011 : 353.56	77,005 : 176.78	38,503 : 88.39
12,960	6,480	173,262 : 397.75	86,631 : 198.88	43,316 : 99.44
14,400	7,200	192,512 : 441.95	96,257 : 220.97	48,128 : 110.5

Notes: In case A, h = 25 ft. and $\phi_e = 20\%$ (or h = 50 and $\phi_e = 10\%$).
In case B, h = 50 ft. and $\phi_e = 20\%$ (text discussion example).
In case C, h = 100 ft. and $\phi_e = 20\%$ (or h = 50 and $\phi_e = 40\%$).
These calculations of aquifer surface area represented are independent of assumptions regarding the geometric shape of the volume of aquifer material from which the pumped waters were withdrawn. The percentage values can exceed 100 because they are relative to one acre of aquifer surface area; this simply means that more than one volume of water corresponding to the volume contained in the aquifer beneath one acre of its surface has been pumped (e.g., two acres worth is 200%).

3.3.5 Casing Storage Effects

In discussing the testing of a well of large diameter, Papadopulos and Cooper (1967) describe the following method to estimate the duration during which stagnant waters standing in the well (casing storage) are drawn on by pumping:

$$t_c = 1440 * (r_c^2 - r_p^2) / T$$

where: t_c is the time during which waters in casing storage can contribute to pumped waters (min.),

r_c is the internal radius of the well casing (ft. or m.),

r_p is the external radius of the pump column (ft. or m.), and

T is the transmissivity (sq.ft. or sq.m.).

Using this equation, examples have been calculated for two hypothetical wells, under a variety of assumed transmissivities. The resulting t_c estimates are listed in Table 3.7. As can be seen by Example 1 in Table 3.7, high yield public water supply wells in moderate to highly transmissive aquifers would draw water exclusively from the aquifer after very brief periods of contributions of water from casing storage. Similarly, Examples 2 and 3 in Table 3.7 show that this would also be true of fairly small pumping wells, such as private domestic wells capable of producing only a few gallons per minute.

Several implications of these hypothetical examples are apparent. Calculations of distance-of-draw estimates need not be viewed with suspicion because of casing storage effects, since errors introduced by

Table 3.7. Examples of Times During Which Waters in Casing Storage Contribute to Pumped Waters

Example 1. A hypothetical well having an inside diameter of 12 inches (r_c = 0.5 ft.) and a pump column outside diameter of 6 inches (r_p = 0.25 ft.)

Transmissivity (ft.^{2}/day)	**Duration of Casing Storage Effects, t_c** (minutes)
10	27
10^2	2.7
10^3	2.7×10^{-1}
10^4	2.7×10^{-2}
10^5	2.7×10^{-3}

Example 2. A hypothetical well having an inside diameter of 6 inches (r_c = 0.25 ft.) and a pump column outside diameter of 4 inches (r_p = 0.167 ft.)

Transmissivity (ft.^{2}/day)	**Duration of Casing Storage Effects, t_c** (minutes)
10	50
10^2	5.0
10^3	5.0×10^{-1}
10^4	5.0×10^{-2}
10^5	5.0×10^{-3}

Example 3. A hypothetical well having an inside diameter of 2 inches (r_c = 0.027 ft.) and a pump column outside diameter of 1 inch (r_p = 0.0069 ft.)

Transmissivity (ft.^{2}/day)	**Duration of Casing Storage Effects, t_c** (minutes)
10	3.0
10^2	3.0×10^{-1}
10^3	3.0×10^{-2}
10^4	3.0×10^{-3}
10^5	3.0×10^{-4}

Notes: These estimates were produced by the method of Papadopulos and Cooper (1967), as discussed in the text.

assuming that all pumped waters come from the aquifer will be insignificant after very brief periods of pumpage. Stagnant waters residing in the well casing will provide only short-lived water quality anomolies. Gibb and others (1981) showed this to be true even for relatively low transmissivity aquifers sampled by very low volume sampling devices.

Scalf and others (1981) note that adequate purging of stagnant waters prior to sampling with bailers and other low volume sampling devices is a necessity. Data from wells which have not been identically purged of stagnant waters are not comparable; use of such data to construct concentration contours (for several wells sampled during a single survey) or historical trends (for a single well sampled at different times) is invalid and misleading. The same logic must be applied to data generated from wells which have been sampled after much longer, but dissimilar, durations of pumpage at identical rates; those data are not directly comparable and their use, too, may result in erroneous conclusions regarding patterns of contaminant concentrations.

3.3.6 Hydraulics and Other Considerations

Estimation of the 'distance of draw' from the total volume pumped is strictly valid only under radial flow conditions, such as occurs in many deep sandstone and limestone aquifers (where natural flow velocities rarely exceed a few feet per year). This estimation procedure cannot be justified when natural ground-water flow velocities are high enough to limit the capture zone of a pumping well to an area of much smaller dimensions than its nominal cone of depression. After that practical limit has been reached, the 'distance of draw' (radial flow) conceptual model is no longer appropriate. The positions and relative rates of the actual ground-water streamlines, resulting from interaction of the natural flow system with the pumping well, then need to be considered in detail. As will be discussed in the subsection on mathematical models, velocity distribution plots should be utilized for such estimates.

Distance of draw estimates are also affected by the manner in which water (and contaminants) move through the unsaturated zone. Contaminant transport by downward percolation from the surface, through intervening unsaturated semipermeable and permeable strata to ground water can be expected to be retarded and redirected laterally. Dependent on the direction of lateral travel, a nearby source of contaminants may have an immediate and serious effect on a well or it may have little or no effect on the well. Of course, the magnitude of lateral travel in the unsaturated zone is extremely important in this regard. This effect is often fairly minor (tens of feet of lateral migration). Due to the rapidly changing characteristics of glacial, fluvial and alluvial deposits, hundreds or thousands of feet of horizontally continuous tight clays or silts are generally rare. Vertical variations are often more exaggerated; stratum thicknesses of inches to a few tens of feet are common.

As a natural consequence of these relatively unresolvable uncertainties, highly accurate distance of draw estimates may be difficult to produce. Therefore, it would be unwise to exclude potential contamination sources from further investigation simply because their physical surface boundaries lie 50 feet, 100 feet, or more beyond the distance of draw calculated for a given contaminant by the methods described here. Numerous uncertainties are involved; the roles that heavy pumping, anisotropy, nonhomgeneity, hydrodynamic dispersion, and chemical and biological interactions play in redistributing contaminants throughout the aquifer have been addressed already. A few other uncertainties deserving of mention, however, are the role precipitation and infiltration play in determining the concentration of contaminants reaching the aquifer by leaching slugs of contaminants from surface sources on a sporadic basis, the role that intermittent pumpage can have on distributing plumes, and the role that well construction plays in determining flowlines (e.g., partial penetration or partial screening effects). These latter uncertainties may have serious consequences if ignored entirely, though their effects may not need to be quantified rigorously (indeed, it may not be possible to do so).

3.3.7 Synopsis of Potential Utility

When drawdown measurements are taken in conjunction with chemical time-series samples, a combined data set results which adequately describes the spatial variablity of the contaminant and the aquifer transmissivity and storage values. This is the kind of information specified by Pfannfuch (1981), among many others, as essential to the efficient design of proper monitoring systems and remedial actions. Even if one cannot be sure that the concentrations determined during chemical time-series sampling are greater or less than the average concentration of the contaminant plume, one may yet attempt to interpret relative arrival times, similarities of contaminant identities, relative concentration ratios and so on.

Moreover, this technique gives the investigator a feel for the statistical uncertainties of contaminant concentrations in the plume he or she is attempting to define. This can be of tremendous help in deciding when a model has been adequately calibrated and when it is predicting with reasonable accuracy. The most serious consequence of excluding considerations of these uncertainties is that doing so may unduly restrict future investigations and conclusions as to potential sources and migration routes of the contamiants.

The noncomparability of raw data collected by point-sampling wells, or by sampling discharging wells that have different flow rates, pumping durations, or construction details has been demonstrated using simple statistical comparisons. The discussion here lends insights into data transformations that may be useful for valid comparisons of such data; e.g., direct weighting by volume of aquifer sampled. Another implication of this discussion is the ability of the user to define minimum purging requirements; e.g., do the contaminant concentrations stabilize after a few borehole volumes have been purged from the well?

3.4 Estimation of Aquifer Coefficients

There are several methods of estimating the transmissivity and the average hydraulic conductivity of an aquifer, and they do not yield comparable information. Transmissivity is the product of the average hydraulic conductivity and the saturated thickness of an aquifer. It is possible, therefore, to estimate transmissivity by first estimating the average hydraulic conductivity of an aquifer, and then multiplying the value obtained by the known thickness of the aquifer (usually obtained from drilling records). If hydraulic conductivity is estimated by laboratory test methods, such as the passage of fluids through cores of the aquifer material that have been placed in permeameter columns, the values obtained will be point estimates only; so that one must consider how much of the aquifer these estimates represent.

3.4.1 Point-Measure Tests

From practical experience, it is easy to see that laboratory estimates of unconsolidated sands and gravels, and clays, are often unreliable. Unconsolidated sands and gravels become disordered and compressed during their extraction from the field and their emplacement in the permeameter columns, which changes their hydraulic properties. Clays can be extracted from the field without too much disturbance, though they suffer compression during emplacement in permeameter columns. The real difficulty with laboratory tests on clays are that one must use unreasonably high hydraulic gradients to get water to flow through the clay sample in a reasonable length of time, and one cannot hope to take a sample that truly represents the amount of secondary permeability present in the clay (caused by minor channels, sand stringers, and other imperfections).

This lack of representativeness accounts for the common observation that a clay found to be 'tight as a drum' by laboratory tests is often found to be 'leaky as a sieve' by field tests. This is very important to understand these days, when estimates of the potential impact of a hazardous waste facility hinge so heavily on the properties of naturally existing and to-be-emplaced clays. On the other hand, laboratory tests on consolidated rock samples, such as limestones and sandstones, often yield reliable estimates of hydraulic conductivity; meaning that they can correlate well with point-measure field-test estimates.

There have been a number of point-measure field tests described in the literature, but the most widely used was introduced by Hvorslev (1951). It consists of displacing a fixed amount of water in a small diameter well or piezometer, and relating the hydraulic response to the properties of the aquifer in the immediate vicinity of the piezometer. As with the laboratory tests, hydraulic conductivity estimates can be obtained by point-measure field tests; however, storage coefficient estimates cannot be obtained by these methods. This is important because both the transmissivity and the storage coefficient are needed for the computations of well hydraulics used to size and position wells for remedial actions.

A variation on the Hvorslev or 'slug' test has been proposed that, theoretically, allows estimation of storage coefficients (Cooper and others, 1967; Papadopoulos and others, 1973). It requires piezometers open to the entire saturated thickness of confined aquifers, and the solution technique is analagous to the Theis solution for drawdown in confined aquifers. Freeze and Cherry (1979) note, however, that estimates of storage coefficients by this method are not practical because the selections of key parameters involved in the solution are ambiguous.

3.4.2 Large Volume Tests

Theis (1935) developed a solution to estimating the drawdown that results from pumping a well in a confined aquifer that has provided the basis for estimating the average transmissivity and storage coefficient of large volumes of an aquifer. The Theis equation, or nonequilibrium or transient equation, relates drawdown to the transmissivity and storage coefficient of confined aquifers as follows:

$$s = (Q / (4 * \pi * T)) * \int_u^\infty (e^{-u} / u)\, du$$

$$= (Q / (4 * \pi * T)) * W(u)$$

where: **s** is the drawdown in the piezometric surface (the level to which confined waters rise, similar to the water table of unconfined aquifers) (ft. or m.),

Q is the discharge rate of the well (cu.ft. or cu.m.),

T is the transmissivity (sq.ft./day or sq.m./day), and

W(u) is the 'well function';

where **u** is defined by:

$$u = (r^2 * s) / (4 * T * t)$$

where: **r** is the radial distance from the pumping well to the point at which drawdown is being measured (ft. or m.),

S is the aquifer storage coefficient (dimensionless),

t is the time since pumping began at rate **Q** (days).

Since the well function, **W(u)**, is an infinite integral, it is necessary to approximate its value with finite expressions. Taylor series expansions have been found to be excellent analytical approximations of the well function:

$$W(u) = -0.5772 - \ln(u) + u - (u^2 /(2 * 2!)) + (u^3 /(3 * 3!))...$$

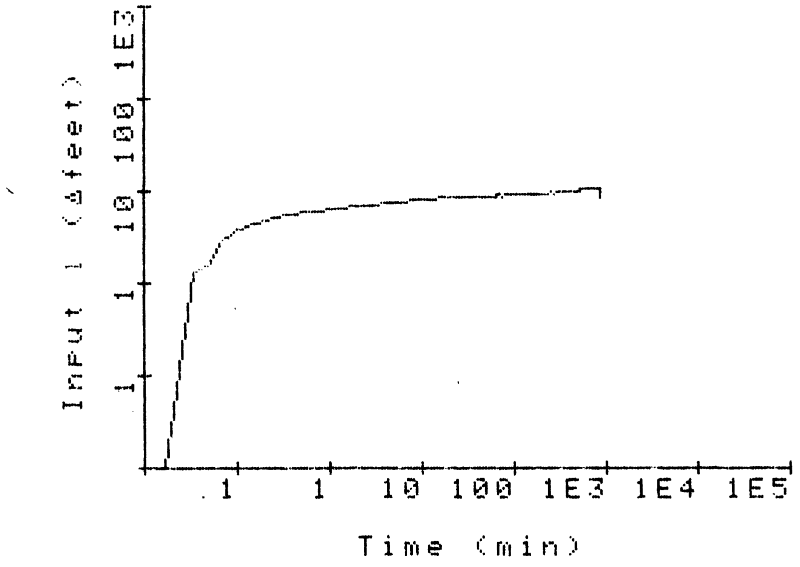
With such approximations of the well function, one can prepare dimensionless plots of **W**(u) versus 1/**u** that are known as 'type curves'. Theis recommended that the solution of his equation be accomplished with type curves by superimposing plots of drawdown versus time on them. The drawdown-time curve resulting from a field test is plotted on log-log paper (Figure 3.24) of the same scale as the type curves. Keeping the coordinate axes of the two sheets parallel, the data curve is overlaid on the type curves until the best fit is found. When this has been done, any point can be chosen as the 'match point' for select paired values of **W(u)** and 1/**u**, and **s** and **t**, to be used in the following expressions for estimation of transmissivity and the storage coefficient:

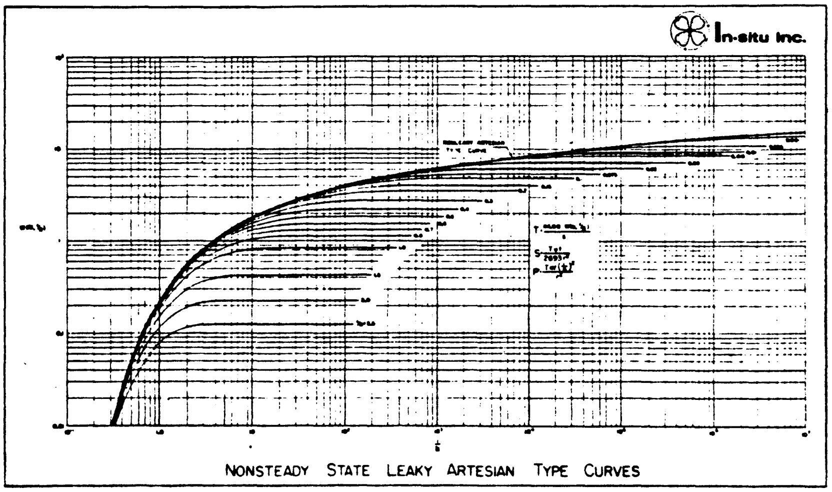
$$T = Q * W\ (u)\ /\ (4 * \pi * s)$$

$$S = (4 * u * T * t)\ /\ r^2$$

The computations are significantly simplified if one chooses the match point to be where **W(u)** and **u** are both equal to one or one-tenth, etc. Matching the data curve to the type curves is the tricky part, as it is as much an art as it is science. Many aquifers do not behave like the fully confined, homogenous, isotropic aquifers of 'infinite areal extent' (meaning no physical boundaries within the pressure influence of the pumping well) that Theis assumed in deriving his equation. Hence, there have been additional type curve solutions devised that treat semi-confined and water-table aquifers, anisotropic conditions and certain boundaries. Lohman (1972) provides an excellent discussion of type curve solutions.

The expanded Taylor series that represents the well function may be truncated without significant loss of accuracy, since the higher order





[From In-situ, Inc. (1983); used with permission (Way, 1986)]

Figure 3.24. Example of Log-Log Drawdown-Time Curve and Corresponding Theis Type Curve of Same Scale

terms have very small numerical values for most practical applications. Cooper and Jacob (1946) recognized that this property of the well function could be exploited to yield a straight-line solution; all higher order terms are dropped so that the solution for drawdown becomes:

$$s = (Q / (4 * \pi * T)) * (-0.5772 - \ln(u))$$

$$= (Q / (4 * \pi * T)) * (-0.5772 - \ln((r^2 * s) / (4 * T * t)))$$

which, when rewritten in common log form, is:

$$s = ((2.3 * Q) / (4 * \pi * T)) * \log((2.25 * T * t) / (r^2 * s))$$

Hence, a plot of drawdown versus time on semi-log paper yields a straight line (Figure 3.25).

The slope of the line in a Cooper-Jacob plot can be used to estimate the transmissivity and the storage coefficient by rearrangement of the above expression; done by noting that when drawdown equals zero, the log term equals one:

$$T = 2.3 * Q / (4 * \pi * \Delta s)$$

$$S = (2.25 * T * t_0) / r^2$$

where: Δs is the drawdown that occurs over one log cycle of time (e.g., from 10 to 100 minutes), and

t_0 is the value of time read off the log axis of the semi-log plot when the straight line is extrapolated back to zero drawdown.

3.4.3 Well Hydraulics

The Theis equation is the most widely used for rapid estimates of drawdown from pumping wells, such as in the design of extraction well systems for removal of contaminant plumes, because of its general

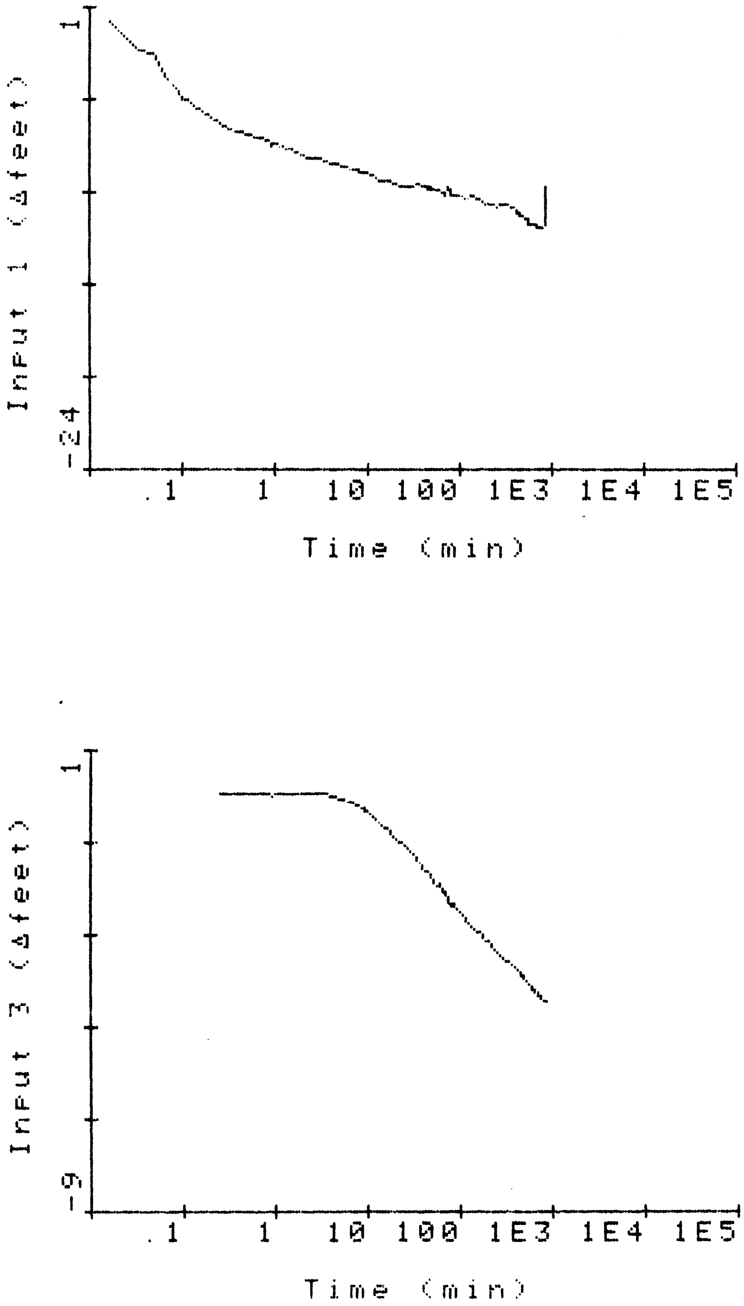


Figure 3.25. Examples of Semi-Log Drawdown-Time Curves

applicability. For example, estimates of drawdown by the Theis method are usually quite good if the drawdown is small compared with the total saturated thickness of the aquifer, which is usually the case unless the observation point is very close to the pumping well or the aquifer has very low transmissivity.

The Theis equation is also quite straightforward mathematically. Table 3.8 was prepared with the Theis equation, using a programmable calculator code (Warner and Yow, 1979); Figures 3.26 - 3.30 are graphical representations of the same information. Examinations of the data in Table 3.8 shows some persistent trends. Since the flowrate of a well is directly proportional to the drawdown, trade-off analyses of the strength of pumpage required to effect a desired hydraulic gradient around a plume (as for hydraulic isolation purposes) are easily made. Order of magnitude increases in the radius of observation cause the same drawdown value to be reached, but not until two log cycles of time later; so that significant hydraulic stresses should only be anticipated at large distances for lengthy operations. The change in drawdown per log cycle of time is constant after only a brief period of pumping for moderate to highly transmissive aquifers, so that it is possible to estimate the long term performance of pumping schemes.

Performing wellfield analyses is fairly straightforward using Figures 26 to 30. If we imagine three wells lying 50, 200, and 750 feet from a common point of interest, for example, the net drawdown caused by pumping each of the three wells at 100,000 cubic feet per day for 100 days from an aquifer with 10,000 square feet per day transmissivity and 0.0001 storage coefficient can be estimated as follows:

Table 3.8. Theis Equation Drawdowns at Various Radii and Times for a Well Pumping 100,000 ft.^{3}/day from an Aquifer with T = 10,000 ft.^{2}/day and S = 0.0001

	TIME (DAYS)						
RADIUS (FEET)	0.01	0.1	1	10	100	1000	10000
1	11.64	13.47	15.30	17.13	18.97	20.80	22.63
2	10.53	12.37	14.20	16.03	17.86	19.70	21.53
3	9.89	11.72	13.55	15.39	17.22	19.05	20.88
4	9.43	11.26	13.10	14.93	16.76	18.59	20.43
5	9.08	10.91	12.74	14.57	16.41	18.24	20.07
6	8.79	10.62	12.45	14.28	16.12	17.95	19.78
7	8.54	10.37	12.21	14.04	15.87	17.70	19.53
8	8.33	10.16	11.99	13.83	15.66	17.49	19.32
9	8.14	9.97	11.81	13.64	15.47	17.30	19.13
10	7.97	9.81	11.64	13.47	15.30	17.13	18.97
20	6.87	8.70	10.53	12.37	14.20	16.03	17.86
30	6.22	8.06	9.89	11.72	13.55	15.39	17.22
40	5.77	7.60	9.43	11.26	13.10	14.93	16.76
50	5.41	7.24	9.08	10.91	12.74	14.57	16.41
60	5.12	6.95	8.79	10.62	12.45	14.28	16.12
70	4.88	6.71	8.54	10.37	12.21	14.04	15.87
80	4.67	6.50	8.33	10.16	11.99	13.83	15.66
90	4.48	6.31	8.14	9.97	11.81	13.64	15.47
100	4.31	6.14	7.97	9.81	11.64	13.47	15.30
200	3.21	5.04	6.87	8.70	10.53	12.37	14.20
300	2.58	4.39	6.22	8.06	9.89	11.72	13.55
400	2.13	3.94	5.77	7.60	9.43	11.26	13.10
500	1.80	3.58	5.41	7.24	9.08	10.91	12.74
600	1.53	3.30	5.12	6.95	8.79	10.62	12.45
700	1.31	3.05	4.88	6.71	8.54	10.37	12.21
800	1.12	2.84	4.67	6.50	8.33	10.16	11.99
900	0.96	2.66	4.48	6.31	8.14	9.97	11.81
1000	0.83	2.50	4.31	6.14	7.97	9.81	11.64
2000	0.17	1.45	3.21	5.04	6.87	8.70	10.53
3000	0.03	0.90	2.58	4.39	6.22	8.06	9.89
4000	0.00	0.56	2.13	3.94	5.77	7.60	9.43
5000	0.0	0.34	1.80	3.58	5.41	7.24	9.08
6000	0.0	0.21	1.53	3.30	5.12	6.95	8.79
7000	0.0	0.12	1.31	3.05	4.88	6.71	8.54
8000	0.0	0.07	1.12	2.84	4.67	6.50	8.33
9000	0.0	0.0	0.96	2.66	4.48	6.31	8.14
10000	0.0	0.0	0.83	2.50	4.31	6.14	7.97

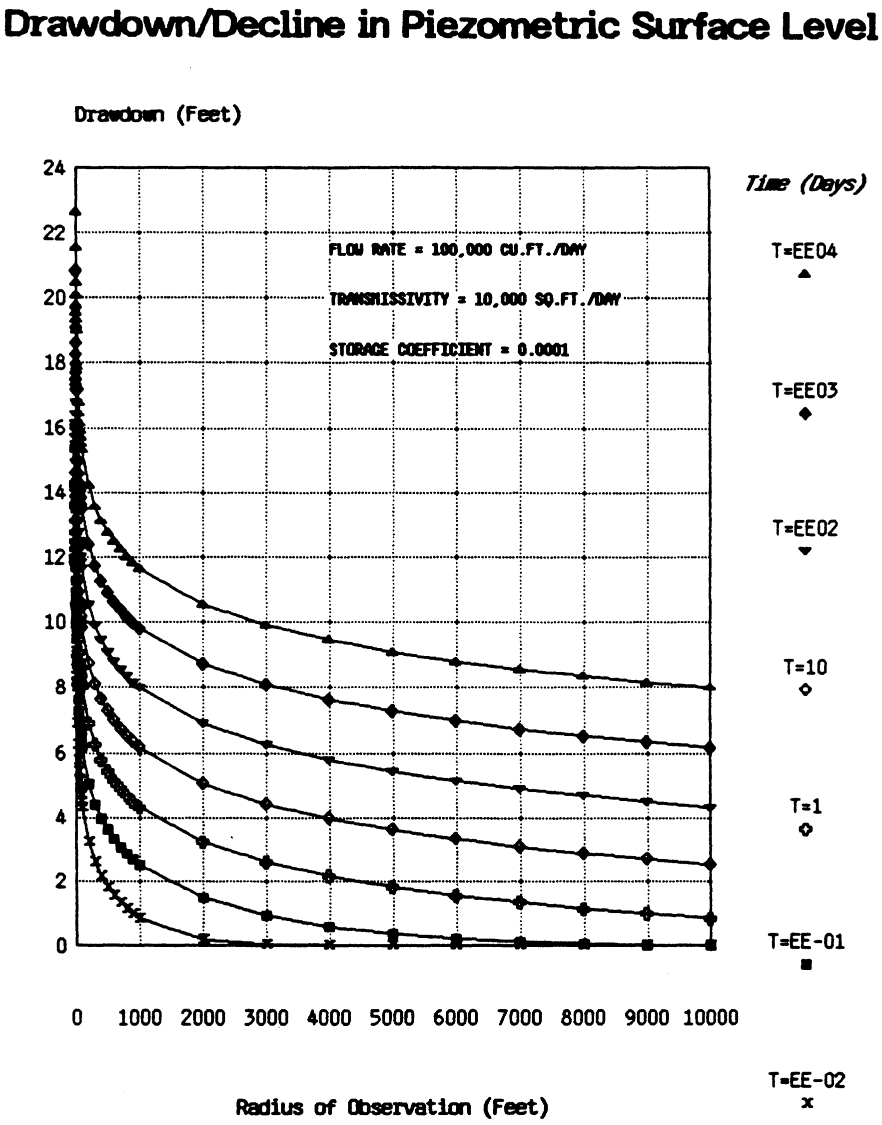


Figure 3.26. Master Plot of Theis Solutions Given in Table 3.8

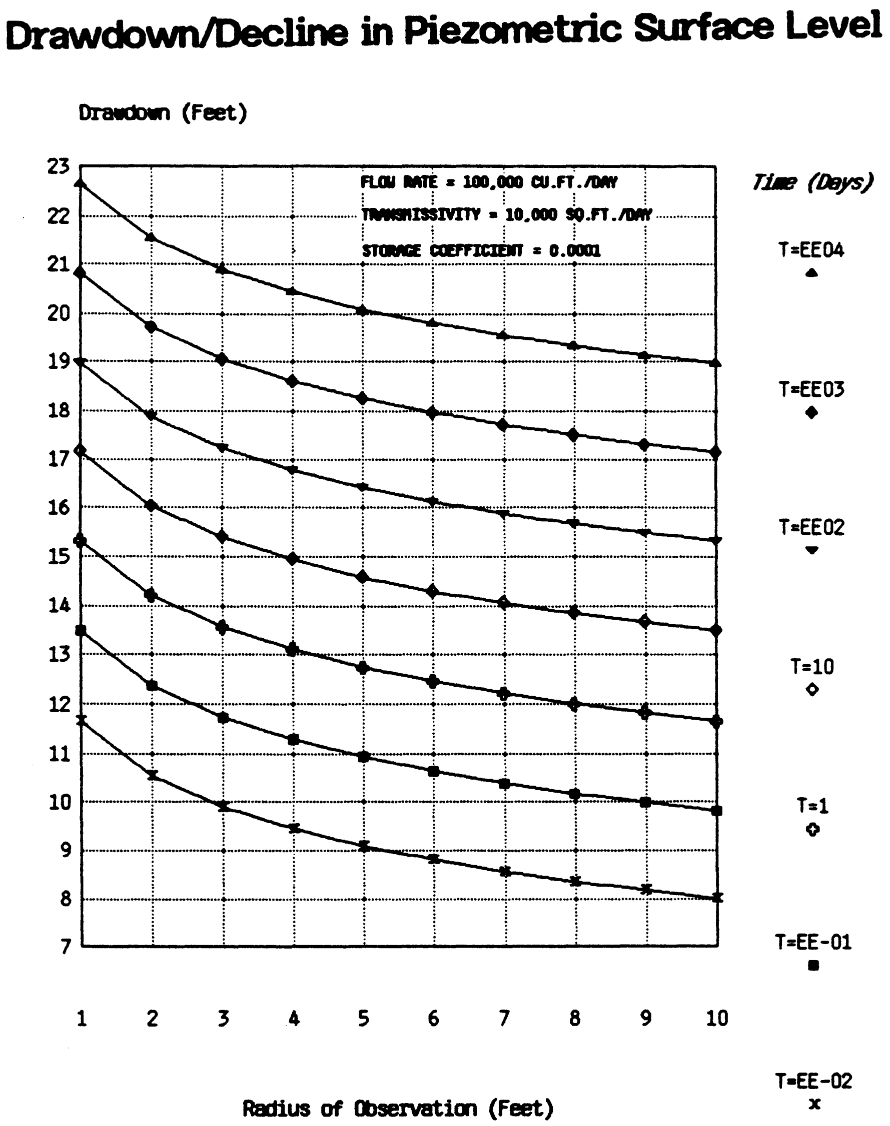


Figure 3.27. High Resolution Plot (for 1 - 10 Foot Radii of Observation) of Theis Solutions Given in Table 3.8

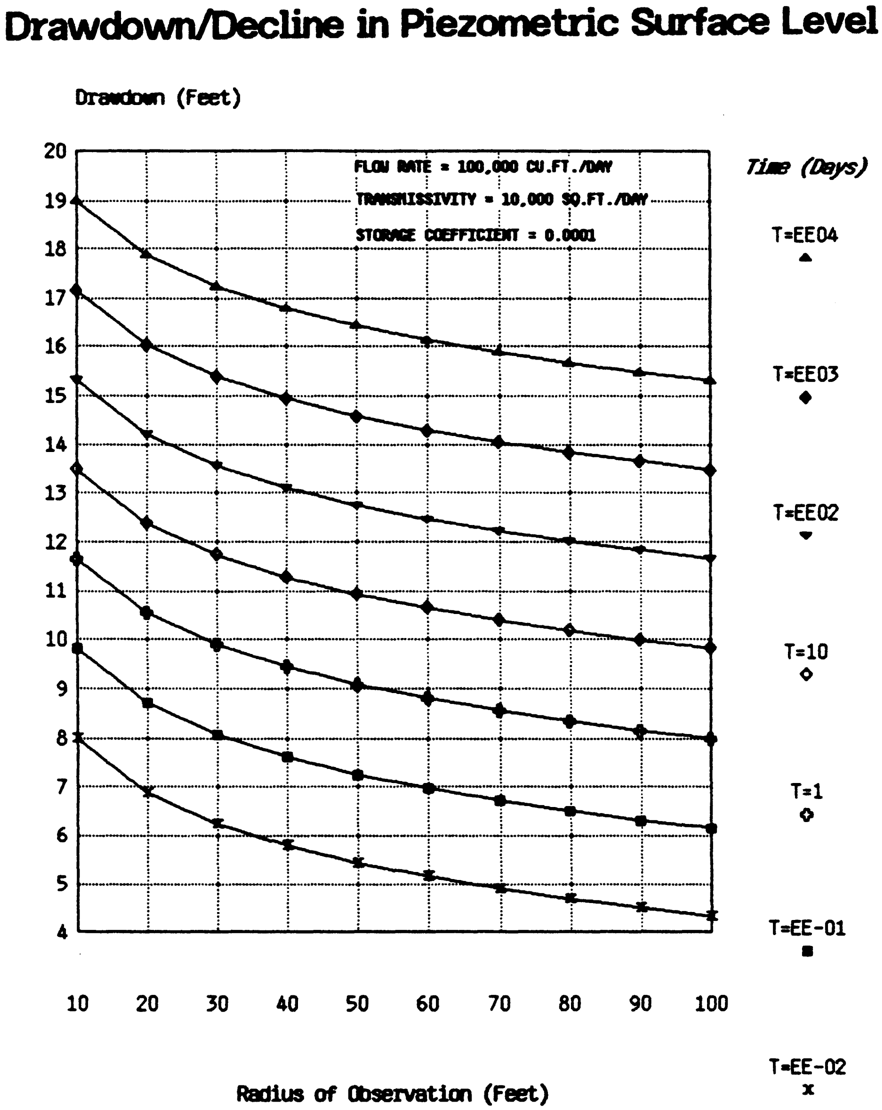


Figure 3.28. High Resolution Plot (for 10 - 100 Foot Radii of Observation) of Theis Solutions Given in Table 3.8

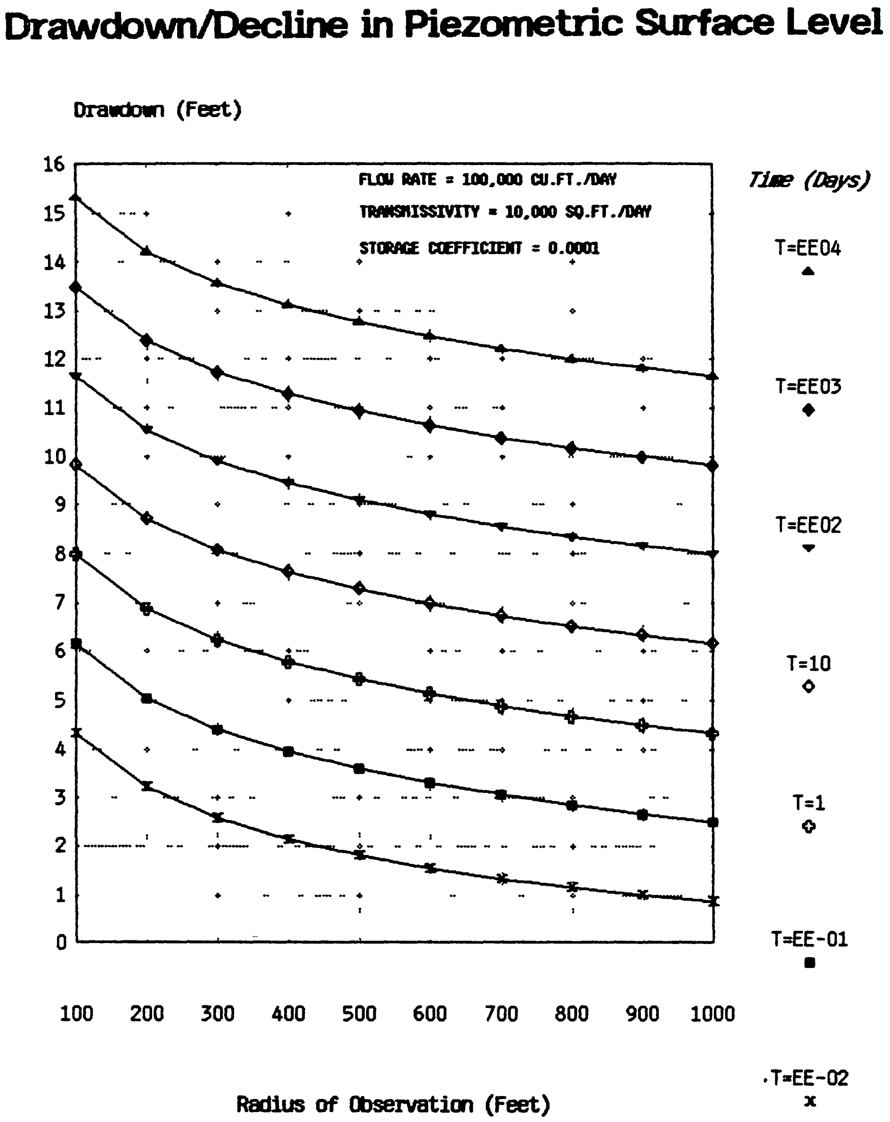


Figure 3.29. High Resolution Plot (for 100 - 1000 Foot Radii of Observation) of Theis Solutions Given in Table 3.8

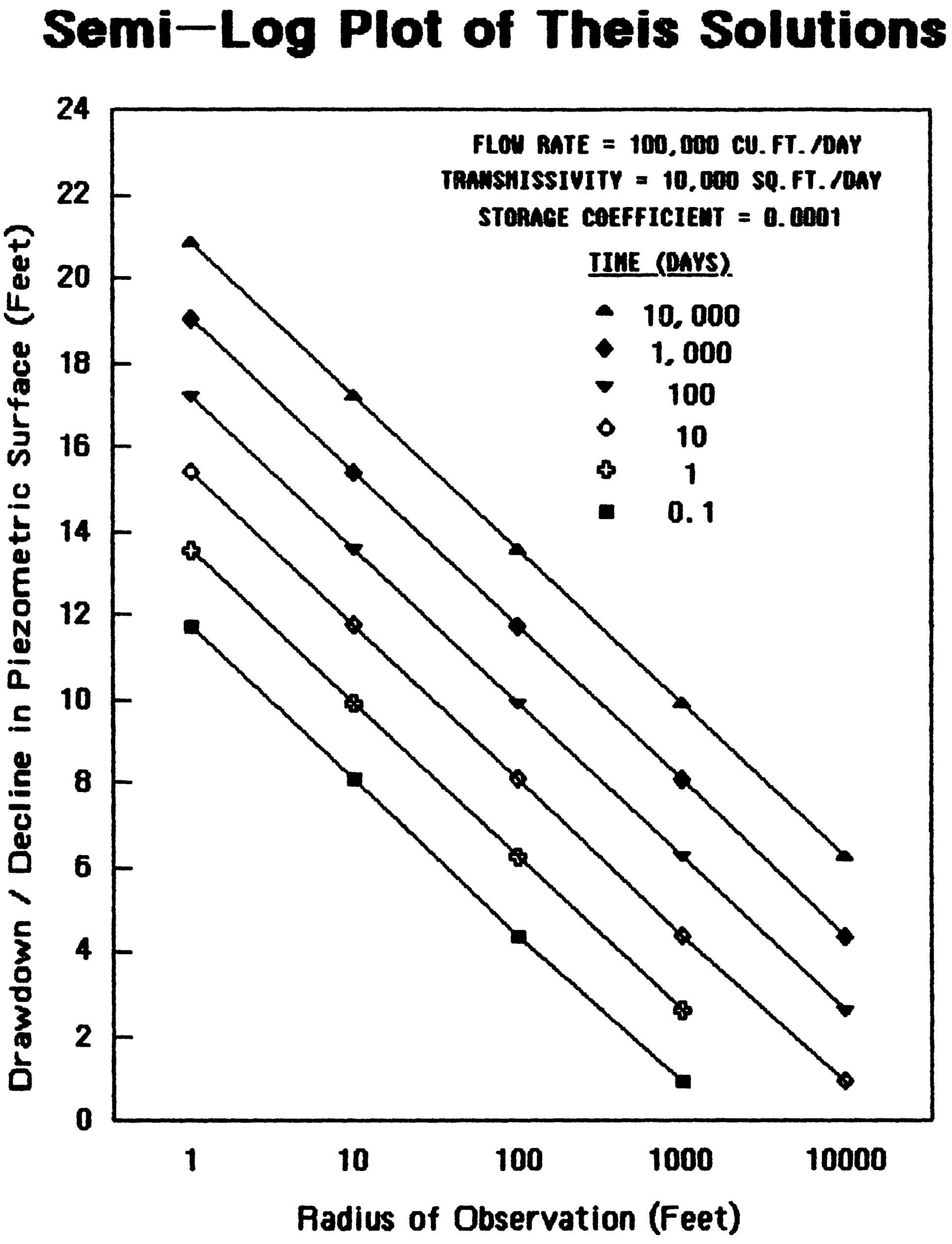


Figure 3.30. Semi-Log Plot of Theis Solutions Given in Table 3.8

1. Read off the drawdown value from the y-axis of Figure 3.28 at the intersection of the 50-foot radius of observation (x-axis) and the 100 day curve; this is about 12.8 feet and is the portion of drawdown caused at the point of interest by the first well.
2. Read off the drawdown value from the y-axis of Figure 3.29 at the intersection of the 200-foot radius of observation and the 100 day curve; this is about 10.5 feet and is the portion of drawdown caused at the point of interest by the second well.
3. Read off the drawdown value from the y-axis of Figure 3.29 at the intersection of an interpolated point on the 100 day curve between the 700- and 800-foot radius of observation lines; this is about 8.5 feet and is the portion of drawdown caused at the point of interest by the third well.
4. Sum the contributions from the three wells to obtain the net drawdown; 12.8 + 10.5 + 8.5 = 32.8 feet of drawdown.

Now modify the example to assume further that the first well pumps only 25,000 cubic feet per day, the second well is an injection well, and the third well has only been pumping for one day:

1. Since flowrates are directly proportional to drawdown for semiconfined to confined aquifers, the first well will generate only one-fourth of the drawdown it did in the previous example, or (25,000/100,000) * 12.8 = 3.2 feet of drawdown.

2. The second well generates a 10.5 foot rise in the piezometric

surface; which is the same magnitude but opposite effect of its role in the previous example.

3. Interpolating between the 700- and 800-foot observation radius lines on the 1 day curve yields an estimate of 4.8 feet of drawdown caused at the point of interest by the third well.
4. The net effect in this example is thus: 3.2 - 10.5 + 4.8 = -2.5 feet of drawdown, or 2.5 feet of increased water level at the point of interest.

3.5 Mathematical Models

Oftentimes in the course of ground-water investigations, water-supply wells are sampled to obtain first-order approximations of the quality of water being delivered to customers. The most common response to showings of unacceptable levels of contaminants is the installation of low-cost, small-diameter nonpumping monitoring wells for the express purpose of estimation of the magnitude and extent of the problem. The differences in construction, operation, and sampling of supply (pumping) wells as opposed to monitoring (nonpumping) wells may result in combined data sets which are confusing to the investigator. A notable exception, of course, is the case where the contaminant of concern has spread ubiquitously throughout the aquifer--a rare occurrence indeed.

Since very limited areal and vertical extent of contaminant plumes is common, combining data from wells of different constructions and flowrates to produce contours of contaminant concentrations for source location or remedial actions could potentially result in poor decisions and wasted time and funds. Unfortunately, such a predicament is all too often encountered. Several recent articles address these points in greater detail (Gibb and others, 1981; Keely, 1982; Keely and Wolf, 1983; Keith and others, 1983; Nacht, 1983; Schuller and others, 1981; and Schmidt, 1977 and 1982). In the present section, it shall be assumed that data are comparable or that they have been appropriately corrected to account for the different sources of data variability. Based on this, several easily mastered methods for rapid estimation of the impact of pumping centers on nearby contaminant plumes are described and illustrated with hypothetical examples.

3.5.1 Manual Plots of Velocity Distributions

As discussed in the preceeding section, the velocity of flow through an aquifer can be represented by rearranging Darcy's Law, which is:

$$Q = K * I * A$$

where:

Q is the volumetric flow rate, (cu.ft./day or cu.m./day),

K is the hydraulic conductivity, (ft/day or m/day),

I is the hydraulic gradient, (dimensionless), and

A is the cross-sectional area through which flow occurs (sq.ft. or sq.m.).

By rearrangement alone, 'Darcy velocity' (V_d) expressions are obtained:

$$Q / A = K * I = V_d$$

But, since the flow actually occurs only through the pores, rather than through the entire cross-sectional area, a slight modification is needed. Division of the Darcy velocity by the effective porosity yields the average seepage velocity:

$$V_d / \emptyset_e = V$$

or, by substitution:

$$(Q / A) / \emptyset_e = V$$

$$(K * I) / \emptyset_e = V$$

Generally one finds the first of these relationships to be most useful for computing the velocity toward a pumping well because **Q** is usually known for the well, and **A** is readily estimated.

Assuming uniformly radial flow toward the well is possible (no blocked sections in the wellscreen), the cross-sectional area through which flow must pass to reach the well is equal to the area of the curved face of an imaginary cylinder of radius r. (That radius can be chosen by the investigator as the distance from the well where the velocity effect is of interest to him or her; the choice is entirely arbitrary, but there is a unique area associated with each radius). The area of the curved face of the imaginary cylinder at that radial distance is:

$$A = 2 * \pi * r * h$$

where **h** is the height of the imaginary cylinder (equal to the effective saturated thickness of the aquifer zone yielding water to the well).

Naturally this implies that there is a distribution of velocities surrounding the pumping well, which increase in magnitude as one gets closer to the well. By substitution of the expression for the area into the appropriate seepage velocity equation, the formula for the velocity due to pumping is obtained:

$$Q / (2 * \pi * r * h * \emptyset_e) = V_{pumping}$$

The-other expression for the seepage velocity form of Darcy's Law is generally employed for estimation of the natural flow velocity:

$$(K * I) / \emptyset_e = V_{natural}$$

This is because the average hydraulic conductivity (**K**) and hydraulic gradient (**I**) are usually known or fairly well estimated for an aquifer, whereas the average bulk flow (**Q**) and cross-sectional area (**A**) of the aquifer are not usually known or estimated accurately. One must estimate the effective porosity ($\emptyset_e$) regardless of the approach adopted.

These simple formulae for $V_{pumping}$ and $V_{natural}$ are quite often all that can be justifiably employed because detailed information on variations in hydraulic conductivity, flow, hydraulic gradient, and so forth are unavailable to the investigator--at least in the initial stages of a contaminant investigation.

Manual plots of the velocity distribution surrounding a pumping well, in the presence of an appreciable natural flow rate and direction, can be readily constructed with the equations just described. The data in Table 3.9 result from such an approach; also incorporated in Table 3.9 is a column listing the theoretical drawdown, calculated by use of the hand-held programmable calculator version of the Theis equation referred to in the preceeding discussions (Warner and Yow, 1979). The important features of the data in Table 3.9 are that velocities from natural flow and pumpage are added together to yield net velocities at distances upgradient ot the well, whereas their differences must be obtained to yield net velocities downgradient of the well. This is quite sensible since the natural flow system is moving waters toward the well on its downgradient side. Figure 3.8 is a graphical presentation of the data in

Table 3.9. Drawdowns and Velocity Toward a Well Pumping 500 Gallons /Minute for 1000 Minutes from an Aquifer with T = 6700 ft^2./day, S = 0.005, h = 100 ft., $\emptyset_e$ = 30%, I = 0.0013

Observation Radius (feet)	Theoretical Drawdown (feet)	Velocity due to Pumping (feet/day)	Net Velocity Upgradient (feet/day)	Net Velocity Downgradient (feet/day)
25	1.19	20.44	23.33	17.55
50	1.04	10.22	13.11	7.33
75	0.94	6.81	9.70	3.92
100	0.88	5.11	8.00	2.22
125	0.83	4.09	6.98	1.20
150	0.78	3.14	6.03	0.25
175	0.75	2.92	5.81	0.03
200	0.72	2.55	5.44	-0.34
225	0.69	2.27	5.16	-0.62
250	0.67	2.04	4.93	-0.85
275	0.65	1.86	4.75	-1.03
300	0.62	1.70	4.59	-1.19
325	0.61	1.57	4.46	-1.32
350	0.59	1.46	4.35	-1.43
375	0.57	1.36	4.25	-1.53
400	0.56	1.28	4.17	-1.61
425	0.55	1.20	4.09	-1.69
450	0.53	1.14	4.03	-1.75
475	0.52	1.08	3.97	-1.81
500	0.51	1.02	3.91	-1.87
*	*	*	*	*
750	0.42	0.68	3.57	-2.21
*	*	*	*	*
1000	0.35	0.51	3.40	-2.38

Note: Positive velocity values indicate flow toward the well, and negative velocity values indicate flow away from the well.

Table 3.9, and can be reviewed to facilitate conceptual appreciation of this discussion.

As can be seen in Figure 3.8 and Table 3.9, at some distance downgradient the pull of waters back toward the well by pumping is exactly countered by the flow away from the well due to the natural flow velocity. Todd (1980) refers to this as the 'stagnation point'; the American Petroleum Institute (1972) refers to it as a 'velocity divide'. Note that the stagnation point/velocity divide occurs well within the cone of depression caused by pumping. This may seem counter-intuitive initially, but calculation of net water surface elevations (by subtraction of drawdown values from prepumping elevations) will confirm that the situation depicted is quite real. This relationship is such that the greater the pumping stress, the farther downgradient the velocity divide occurs (for a given natural flow velocity). Conversely, the greater the natural flow velocity, the closer the divide comes to the pumping well (for a given pumping stress).

It is much more efficient to solve directly for the distance to the stagnation point than it is to construct plots like Figure 3.8. One abides by the definition of the stagnation point and sets the expression for $\mathbf{V_{pumping}}$ equal to the value of $\mathbf{V_{natural}}$:

$$\mathbf{V_{pumping} = V_{natural}} \qquad \text{(definition of stagnation)}$$

$$\mathbf{Q / (2 * \pi * r * h * \emptyset_e) = V_{natural}} \qquad \text{(substitution)}$$

and then rearranges this to solve directly for the distance to the stagnation point $\mathbf{r_{sp}}$:

$$\mathbf{r_{sp} = Q / (2 * \pi * h * \emptyset_e * V_{natural})}$$

Using the data from Table 3-9 (for the graphical comparison, see Figure 3-8):

$$r_{sp} = (9.63 \times 10^4 \text{ ft}^3\text{/day}) / (2 * \pi * 100 \text{ ft} * 0.30 * 2.895 \text{ ft/day})$$

$$r_{sp} = 176.5 \text{ ft} \quad \text{or } 53.8 \text{ m}$$

Todd (1980) notes that the maximum width of the capture zone occurs slightly upgradient of the well and is equal to 2π times the distance to the stagnation point:

$$W_{max,cz} = 2 * \pi * r_{sp}$$

With homogenous and isotropic aquifer conditions, the capture zone is symmetrically disposed about a line parallelling the natural flow direction and running through the well. Hence, contaminated waters lying just beyond π times the stagnation distance to either side of the well are not drawn into the well (for the preceding example: π x 176.5 ft = 554 ft or 169 m). Again, this occurs despite the fact that significant drawdowns are felt there (0.5 ft or 15 cm; Table 3.9). These boundaries define the areal limits of what is referred to as the 'capture zone' of the well. Only for the extremely rare case of zero natural flow velocity are the areal boundaries of the capture zone and the cone of depression everywhere identical.

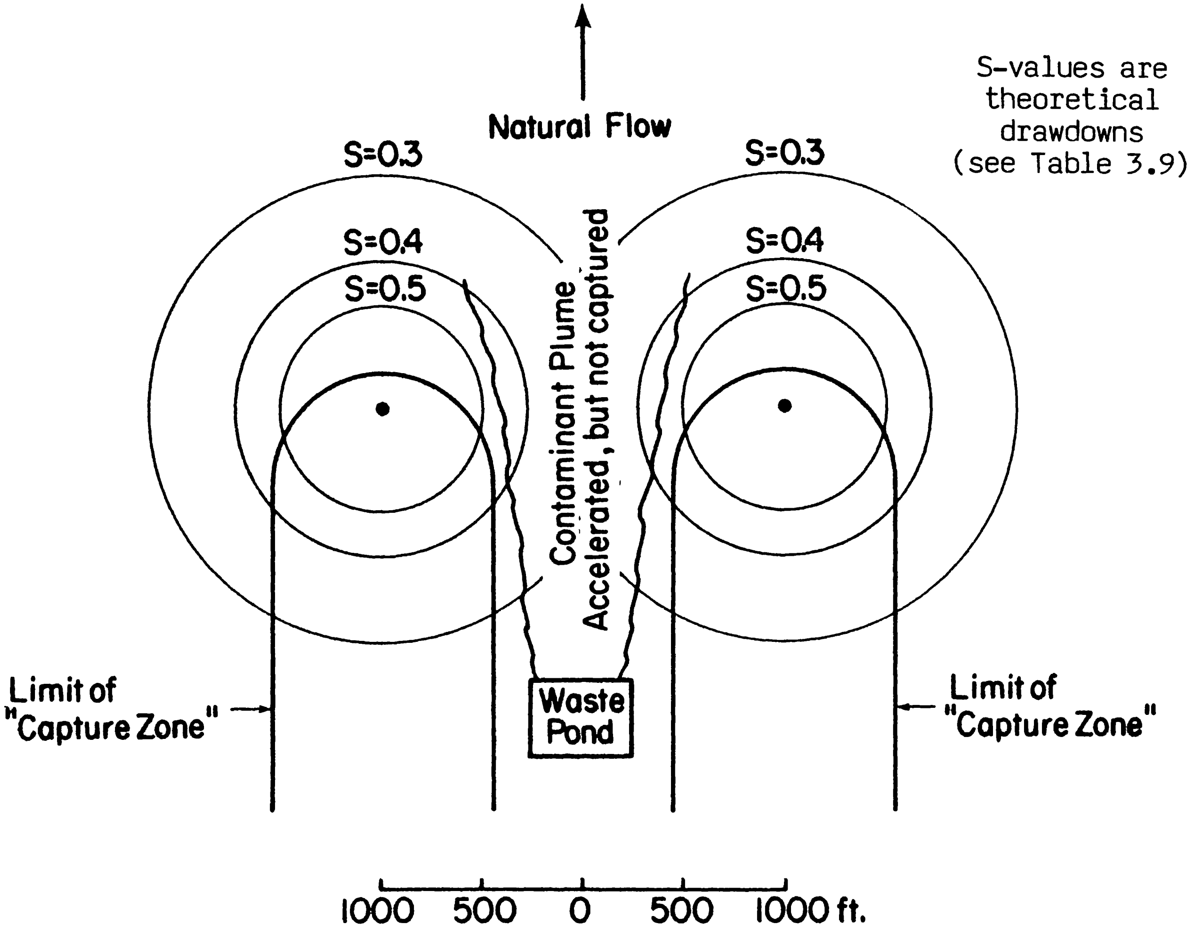
The importance of performing these calculations can hardly be overemphasized for ground-water contamination investigations. Using such calculations it is no great task to show that a line of wells designed to stop the advance of a contaminant plume due to a fairly large natural flow velocity may fail miserably, despite the fact that their adjacent cones of depression overlap (an almost sacred benchmark for field practitioners). Figure 3.31 illustrates such a situation, using selected data from Table 3.9.

It is also important to note that this is not an earth-shattering discovery. Virtually all contaminant transport codes are based on the calculation of a velocity field, so that use of such codes need not be viewed with suspicion from that standpoint. However, it is the authors' contention that because the vast majority of ground-water contamination investigations are initiated, performed, and concluded without the use of numerical transport models, and because qualitative decision benchmarks such as 'overlapping cones of depression' are heavily employed, it is imperative that velocity plots now be emphasized.

3.5.2 Radial Flow Time-Series Model (RT)

Major impediments to the widespread use of sophisticated contaminant transport models include their general reliance on advanced mathematics, the need for large computing systems and programming skills, and the tedious selection and construction of appropriate grids. In order to circumvent the most undesirable of these characteristics, two nondispersive transport codes are presented here which rely on the simple velocity expressions discussed in the preceding section. The examples employed here to illustrate these codes were produced with very short FORTRAN programs, which are currently available (Javandel and others, 1984).

The radial flow time-series model, RT, is particularly useful for estimation of impacts to a pumping well from nearby contaminant sources. Since this code ignores regional flow, it is not as detailed as might be



S-values are theoretical drawdowns (see Table 3.9)

Notes: Same as notes for Figure 3.8 and Table 3.9.

Figure 3.31. Contaminant Plume Escaping Between Two Wells That Have Overlapping Cones of Drawdowns

required for many complete analyses, but its simplicity and brevity makes it attractive for rapid estimation purposes (RESSQ, the code described in the next subsection, incorporates regional flow). One may use RT with confidence for such situations as the combination of a large pumping well and a low regional (natural) flow velocity. Caution must be exercised when using RT for field problems where low pumping stresses are combined with appreciable natural flows because substantial errors may result. RT is capable of providing several useful plots:

(1) time-concentration data,
(2) radial distance-concentration data,
(3) specified point ($\mathbf{x},\mathbf{y}$)-concentration data, and
(4) selected concentration contours.

The primary situation examined here by RT is one where a pumping well is surrounded by several observation wells, some of which are being impacted by a spreading contaminant plume. Of great interest are the changes in levels of contaminants at the observation wells and the pumping well as pumping progresses; these concentration-time patterns will yield substantial clues as to the spatial distribution of the contaminants. This technique of correlating time-series data to spatial distributions has been developed from the techniques described by Keely (1982).

Because of the radial flow situation addressed by RT, it is useful to slightly modify the expression for $\mathbf{V}_{\mathbf{pumping}}$ given earlier. In radial coordinates, it is:

$$\mathbf{V}_{\mathbf{pumping}} = \mathbf{V}_{\mathbf{radial}} + \mathbf{V}_{\mathbf{beta}} = \mathbf{V}_{\mathbf{r}} + \mathbf{V}_{\beta}$$

where V_r and $V_ß$ are the radial and angular components of velocity in the radial coordinate system (r, ß), analogous to the **x** and **y** velocity components of the Cartesian system (**x**, **y**). Substitution of the expression for $V_{pumping}$ then gives:

$$V_{pumping} = Q / (2 * \pi * r * h * \emptyset_e) = V_r + V_ß$$

For a pumping well at the origin (0,0) of the plot to be constructed, $V_ß$ equals zero by symmetry, so $V_{pumping} = V_r$. Hence, the expression for V_r at some radial distance (**r**) from the pumping well in radial coordinates is the same as $V_{pumping}$ calculated previously by considering the distance to the edge of an imaginary cylinder around the well:

$$V_r = Q / (2 * \pi * r * h * \emptyset_e)$$

Recognizing that the velocity along any radial flowline (V_r) is the result of the change in distance being divided by the change in time (**dr/dt**):

$$V_r = dr/dt$$

a substitution can be made:

$$dr/dt = Q / (2 * \pi * r * h * \emptyset_e)$$

The 'distance of draw' calculations introduced earlier in this section were a special case of this more general expression; there, the distance travelled from the onset of pumpage (**dr**) was found by multiplying the right hand side of this expression by the duration of pumpage (**dt**). This general expression leads directly to equations useful for estimating the distance travelled by contaminant fronts that are not necessarily adjacent to the well, and the time required to move from one arbitrary radial distance to another.

To arrive at those equations requires integration of this differential expression over distance and time, respectively; which requires only the most straightforward rules of integration:

1. Constants in the equation are unaffected.
2. Open integrals imply simple subtraction of the maximum and minimum values (in this case, the time of pumping, or $t - t_0$).
3. A variable (here, radial distance) is raised to the next power and divided by a value equal to that power (e.g., r becomes $r^2/2$).

Hence, integration of the radial velocity expression gives:

$$(r^2/2 - r_0^2/2) / (t - t_0) = Q / (2 * \pi * h * \emptyset_e)$$

which can be readily rearranged to solve for the distance traveled from any arbitrary point (r_0) during a specific time ($t - t_0$):

$$r = (r_0^2 + (Q * (t - t_0) / (\pi * h * \emptyset_e)))^{1/2}$$

Likewise, it can be rearranged to solve for the time required to travel a specific distance ($r - r_0$):

$$t = t_0 + (((r^2 - r_0^2) * \pi * h * \emptyset_e) / Q)$$

These two equations form the basis for calculations performed by RT.

For example, if an observation well is located at a distance r_1 from the pumping well, then a time series of contaminant concentration measurements at that well taken at t_1, t_2, t_3, . . . , t_n, will yield the corresponding locations of r_2, r_3, . . . , r_{n+1} for those concentrations at any given time. Hence, assuming that the concentration distribution of a given solute in an aquifer is not uniform, the time-series data from a given well can be mapped out into the aquifer to produce a 'snapshot' of the spatial contaminant concentration distribution, along the radius between the observation well and the production well at various times. By using the observation wells in several directions from the pumping well, an areal picture of the contaminant concentration in the aquifer at various times can be determined.

To illustrate this situation, RT was used to create the sequence of plots shown in Figure 3.32, which run from prepumping to a little more than one day's pumping. Each of the scatter points is brought closer to the origin (0,0) by the pumping well located there. Alternatively, RT can be used to generate contours of relative concentration, such as shown in Figure 3.33.

Perhaps the most powerful outputs generated by RT, however, are the individual relative concentration-time plots for any of the six monitoring/observation wells or the pumping well (Figure 3.34). It should come as no surprise that these patterns of contaminant arrival look like the breakthrough curves generated during tracer experiments; the fundamental laws and the field design are the same. The noticeable difference is that the low level leading edge is absent from the early time portions of the plots because dispersive effects are not accounted for by RT.

An invaluable variation on this presentation of relative concentration-time plots is also output by RT (Figure 3.35). Relative concentrations are plotted versus distance for a select number of times of interest, generating a family of curves. Thus one can examine the relative concentration along a selected radial and readily observe the

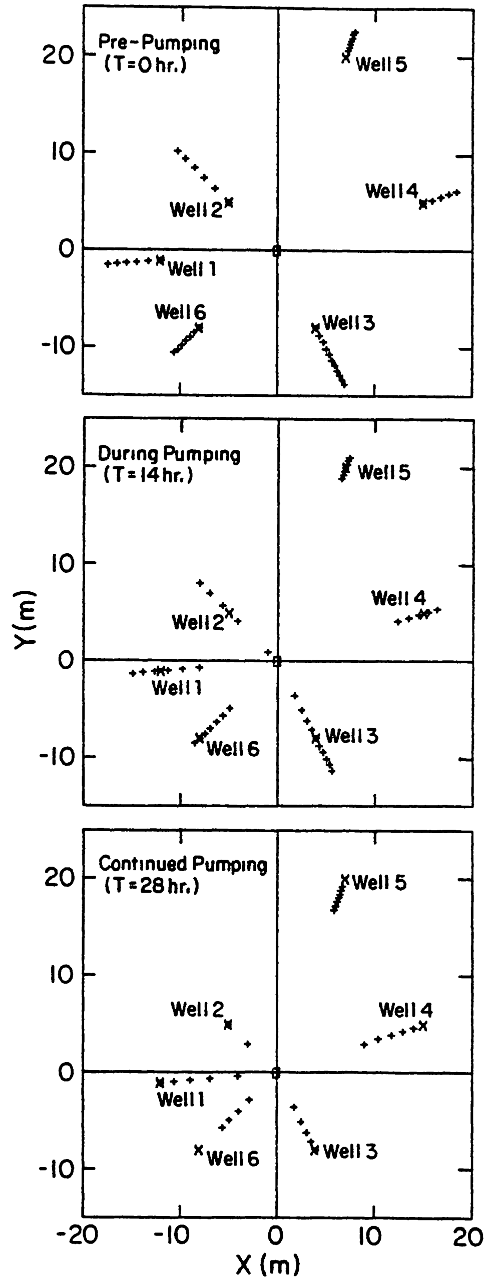


Figure 3.32. Spatial-Distribution Scatter Maps for Various Durations of Constant Pumpage, Produced by RT Model

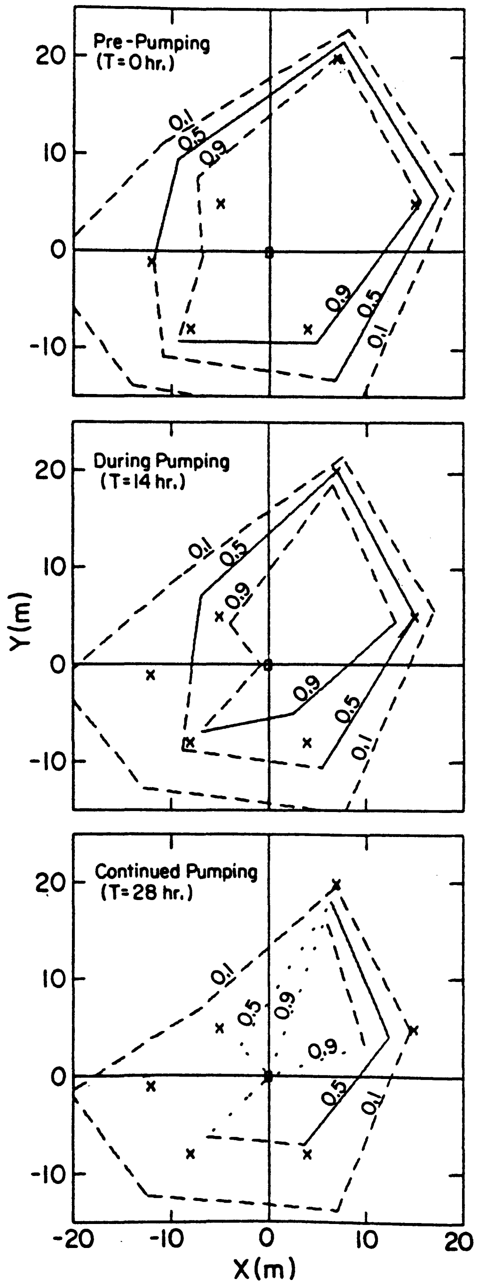


Figure 3.33. Concentration-Distribution Contour Maps for Various Durations of Constant Pumpage, Produced by RT Model

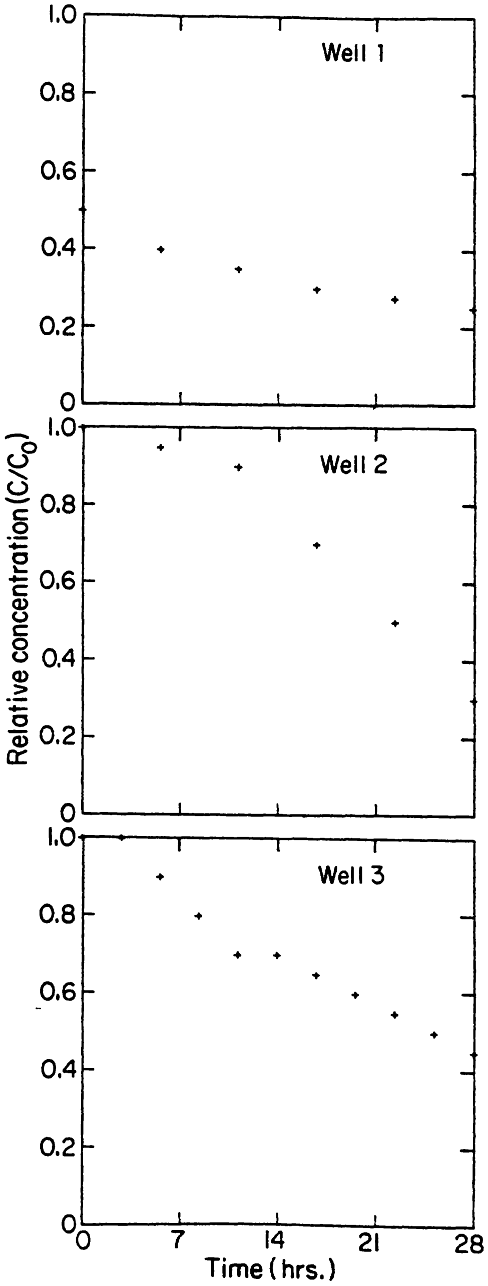


Figure 3.34. Relative Concentration-Time Plots for Various Durations of Constant Pumpage, Produced by RT Model

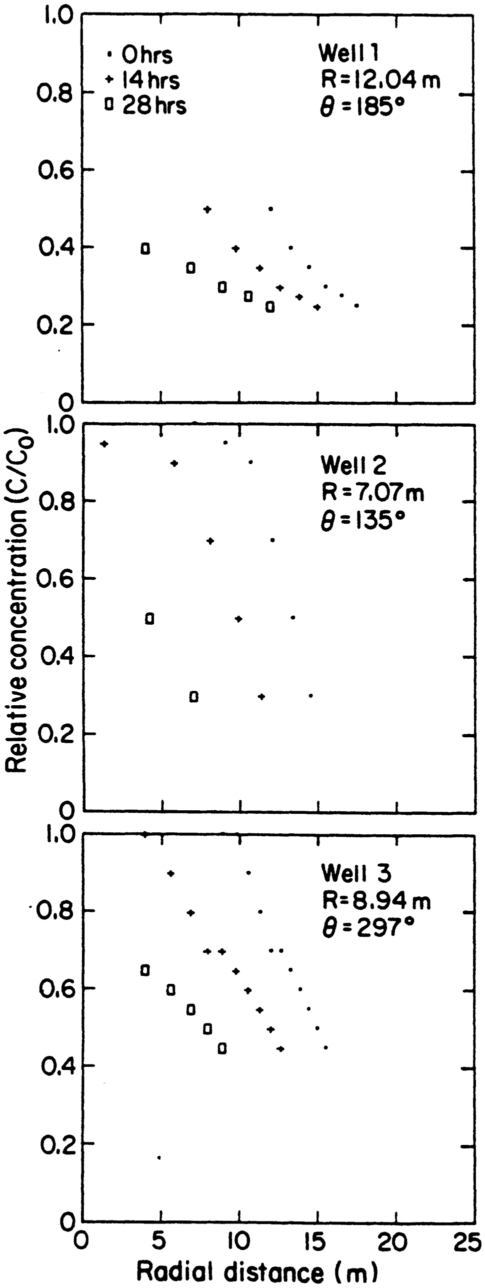


Figure 3.35. Relative Concentration-Radial Distance Plots for Various Durations of Constant Pumpage, Produced by RT Model, for the Case Without Attenuation (R = 1.0)

manner in which this relationship changes with time. In the example shown in Figure 3.35, it can be seen that the farther from the pumping well, the less the disturbance of the contaminant plume. As one gets closer to the pumping well, the greater the disturbance, so that the overall relative concentrations are lowered rapidly and the length of the plume expands considerably. This kind of graphical presentation underscores the need to plot velocity distributions to estimate the impact of pumping centers on plume movement. Plume travel time estimates based solely on an average velocity will be much greater than they ought to be, giving planners a false sense of security or lack of urgency.

Because of retardation/attenuation or degradation of some contaminannts by physical, chemical, or biological interactions, the velocities at which a contaminant species are transported through the subsurface may be substantially less than the seepage velocity. The retardation factor (**R**) can be incorporated into the velocity calculations of the water front movement ($\mathbf{V_I}$) to give the velocity of the contaminant ($\mathbf{V_C}$):

$$\mathbf{V_C = V_I / R}$$

where **R** is again equal to the ratio of the seepage velocity to the contaminant velocity ($\mathbf{R = V_I / V_C}$), as introduced in the preceeding section. Figure 3.36 is a replotting of the example in Figure 3.35, but with **R** = 1.5.

3.5.3 RESSQ Model

While the brevity and simplicity of RT make it a useful tool for rapid assessment of radial flow situations, it does lack the ability to deal

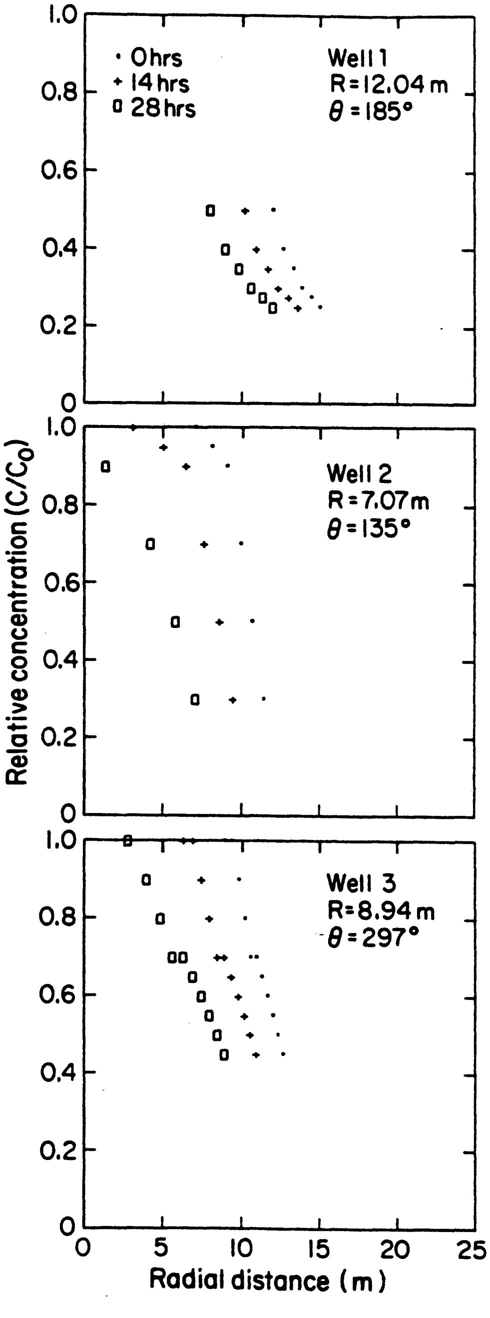


Figure 3.36. Relative Concentration-Radial Distance Plots for Various Durations of Constant Pumpage, Produced by RT Model, for a Case with Moderate Attenuation (R = 1.5)

with the effect on the velocity distribution caused by appreciable regional/natural flow. As was shown earlier in this section (manual plotting techniques for velocity distributions), the effect of a moderate natural flow velocity can be quite important.

RESSQ (REgional Source Sink flow/Q model; pronounced 'rescue') is an expanded version of RT, capable of incorporating the natural flow velocity and also capable of simulating more complex situations where several pumping wells and contaminant sources need to be evaluated simultaneously. As such, it has been constructed so that all inputs and outputs are geared to a Cartesian (**x,y**) coordinate system (see Figure 3.37). The expression for velocity due to a pumping well was given previously as:

$$V_r = Q / (2 * \pi * r * h * \emptyset_e)$$

where **r** was the distance from the origin (0,0) to the edge of an imaginary cylinder surrounding the well (radial flow assumption). It is useful to create a more generalized expression in terms of actual **x**- and **y**-coordinates. The easiest approach is to use the origin (0,0) as a reference point and employ the Pythagorean theorem of geometry for right triangles:

$$r^2 = x^2 + y^2$$

Here, the right triangle has it short sides parallel to the positive **x**- and **y**-axes, and its hypotenuse (of length **r**) defining the line connecting the origin (0,0) to the point of interest (**x,y**). For a pumping well located at (x_o, y_o), the pumping velocity at the point of interest (**x,y**) has **x**- and **y**-components given by:

$$v_x = (Q/(2 * \pi * h * \emptyset_e)) * ((x - x_o) / ((x - x_o)^2 + (y - y_o)^2)$$

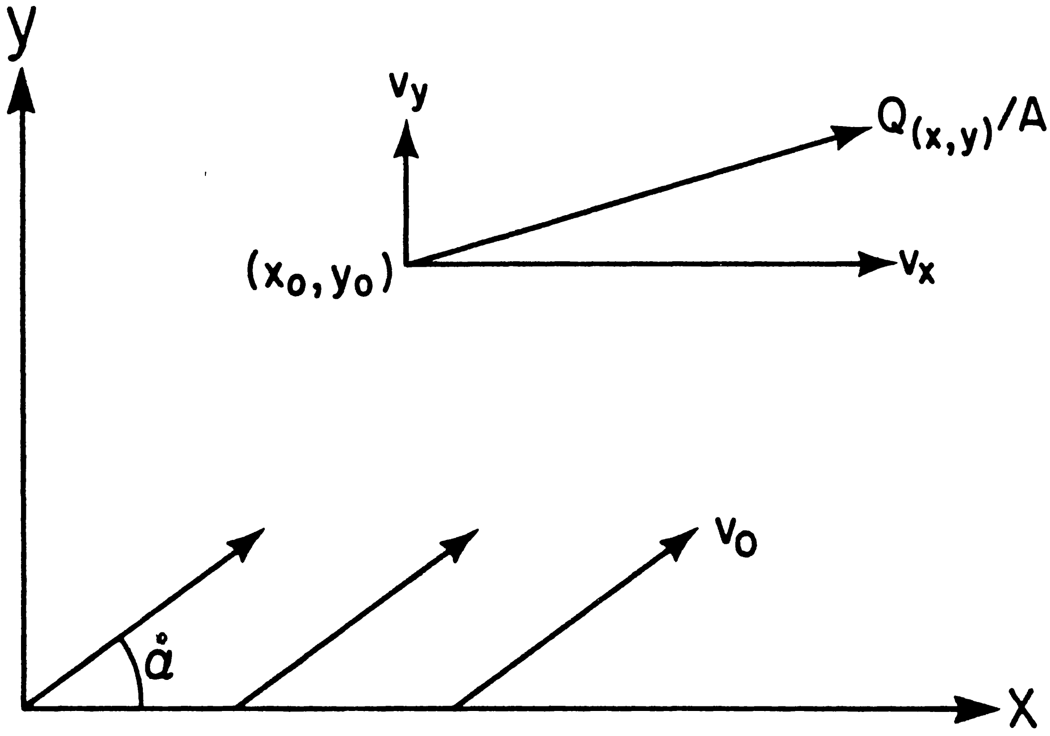


Figure 3.37. Cartesian System Showing X,Y-Components of Pumping Velocity, and Regional Velocity at Angle å

and

$$v_y = (Q/(2 * \pi * h * \emptyset_e)) * ((y - y_0) / ((x - x_0)^2 + (y - y_0)^2)$$

The **x** and **y** components of the natural flow velocity V_n, which has a direction at an angle, **å**, from the x-axis, are:

$$V_x = V_n * \cos å$$

and

$$V_y = V_n * \sin å$$

If one considers a waste-water injection well with an injection rate of 220 gal/min (50 cu.m./hr) located at point **A** in Figure 3.38, which is suspected of potentially contaminating a water-supply well that produces at an identical flowrate and is located 2784 feet (848.5 meters) away, at point **B**, the prime questions to be answered are:

1. What do the flow patterns of the system look like?
2. Where will the injected waste-water front be after certain time periods (e.g., 0.5, 2, and 4 years)?
3. How long does it take for the injected waste water to reach the water-supply well?
4. How does the contaminant concentration vary at the water-supply well?

Based on local geology, it has been estimated that the effective porosity ($\emptyset_e$) of the aquifer is 25 percent.

As a first cut, analogous to the simple approach in RT, the regional/natural flow velocity will be neglected. Using RESSQ, Tables 3.10 and 3.11 are generated. Table 3.10 lists the arrival times at the pumping well of the injected waste waters flowing along flowlines between

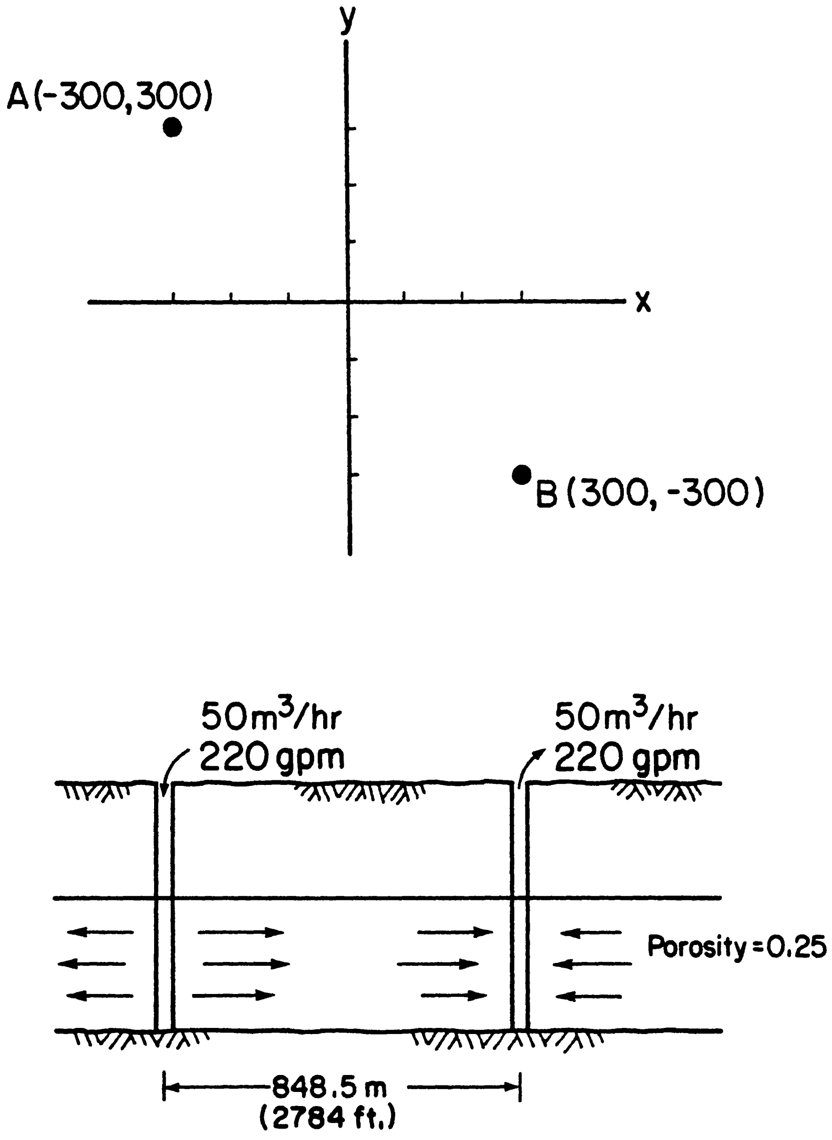


Figure 3.38. Plan and Cross-Sectional Views for the RESSQ Example Discussed in the Text

Table 3.10. Time of Arrival of Flowlines for RESSQ Example Discussed in the Text: the Case Without Regional Flow

Injection Well Flowline Number	Captured by the Production Well?	Arrival Time (years)	Departure Angle of Flowline (ß)
1	yes	5.6	0
2	yes	6.2	8
3	yes	7.0	16
4	yes	8.1	24
5	yes	9.5	32
6	yes	11.6	40
7	yes	14.4	48
8	yes	18.5	56
9	yes	24.6	64
10	yes	34.3	72
11	yes	50.4	80
12	yes	79.3	88
13	yes	137.0	96
14	no	---	104
15	no	---	112
16	no	---	120
17	no	---	128
18	no	---	136
19	no	---	144
20	no	---	152
21	no	---	160
22	no	---	168
23	yes	118.3	176
24	yes	70.3	184
25	yes	45.5	192
26	yes	31.4	200
27	yes	22.8	208
28	yes	17.3	216
29	yes	13.6	224
30	yes	11.0	232
31	yes	9.1	240
32	yes	7.8	248
33	yes	6.8	256
34	yes	6.0	264
35	yes	5.4	272
36	yes	5.0	280
37	yes	4.7	288
38	yes	4.5	296
39	yes	4.4	304
40	yes	4.3	312
41	yes	4.3	320
42	yes	4.4	328
43	yes	4.5	336
44	yes	4.8	344
45	yes	5.1	352

Table 3.11. Relative Concentration at Pumping Well for RESSQ Example Discussed in the Text: the Case Without Regional Flow

Injection Well Flowline Number	Departure Angle of Flowline (ß)	Arrival Time (years)	% Relative Concentration
40	312	4.3	2.22
41	320	4.3	4.44
39	304	4.4	6.67
42	328	4.4	8.89
38	296	4.5	11.11
43	336	4.5	13.33
37	288	4.7	15.56
44	344	4.8	17.78
36	280	5.0	20.00
45	352	5.1	22.22
35	272	5.4	24.44
1	0	5.6	26.67
34	264	6.0	28.89
2	8	6.2	31.11
33	256	6.8	33.33
3	16	7.0	35.56
32	248	7.8	37.78
4	24	8.1	40.00
31	240	9.1	42.22
5	32	9.5	44.44
30	232	11.0	46.67
6	40	11.6	48.88
29	224	13.6	51.11
7	48	14.4	53.33
28	216	17.3	55.56
8	56	18.5	57.78
27	208	22.8	60.00
9	64	24.6	62.22
26	200	31.4	64.44
10	72	34.3	66.67
25	192	45.5	68.89
11	80	50.4	71.11
24	184	70.3	73.33
12	88	79.3	75.56
23	176	118.3	77.78
13	96	137.0	80.00

Note: Because the injected flow is represented by 45 flowlines, each carries 1/45th or 2.22% of the contaminated injection fluid.

the two wells, depicted graphically by RESSQ in Figure 3.39. Note that nine of the flowlines carrying the contaminants have not reached the water-supply well during the specified period of study which was arbitrarily chosen to be 200 years. The angle (**B**) at which each flowline leaves the injection well (measured counterclockwise from the positive **x**-axis) is also shown in Table 3.10. Note that flowlines #40 and #41 leave the injection well at angles of 312 and 320 degrees, respectively, and are the first to arrive at the water-supply well--which agrees with one's intuitive expectations. The time of arrival of these two flowlines is 4.3 years; the water-supply well is affected rather quickly in terms of a normal operational lifetime for the injection well (10-20 years).

Table 3.11 presents the time variation of concentration at the water-supply well. Each of the 45 flowlines emanating from the injection well represents 1/45 of the total injection rate, so mixing of the waste water carried by each flowline with other unaffected waters drawn on by the water-supply well increases the relative concentration of the waste-water contaminants by 1/45 or 2.22 percent. For the study period examined in this example, a maximum of 36 flowlines reach the water-supply well from the waste-water injection well, resulting in a maximum relative concentration of 80 percent. Hence, if the injected waste water was laden with 200 ppm of contaminant XYZ, then 0.80 * 200 ppm, or 160 ppm is the maximum concentration of contaminant XYZ in the water discharged from the supply well. RESSQ is capable of presenting these data graphically, as shown in Figure 3.40. Alternatively, RESSQ can be used to display selected contours, such as the position of the waste-water front after 0.5, 2, and 4 years, as shown in Figure 3.41.

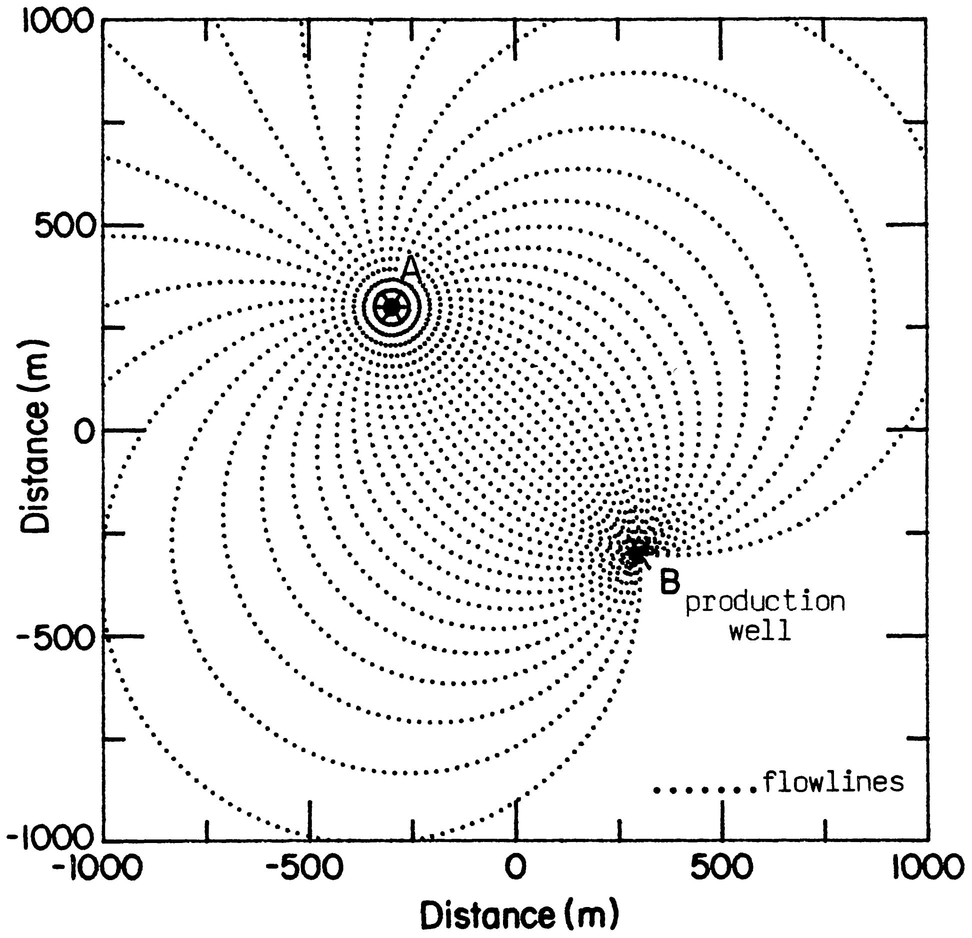


Figure 3.39. Flowline Plot for the RESSQ Example Discussed in the Text: the Case Without Regional Flow

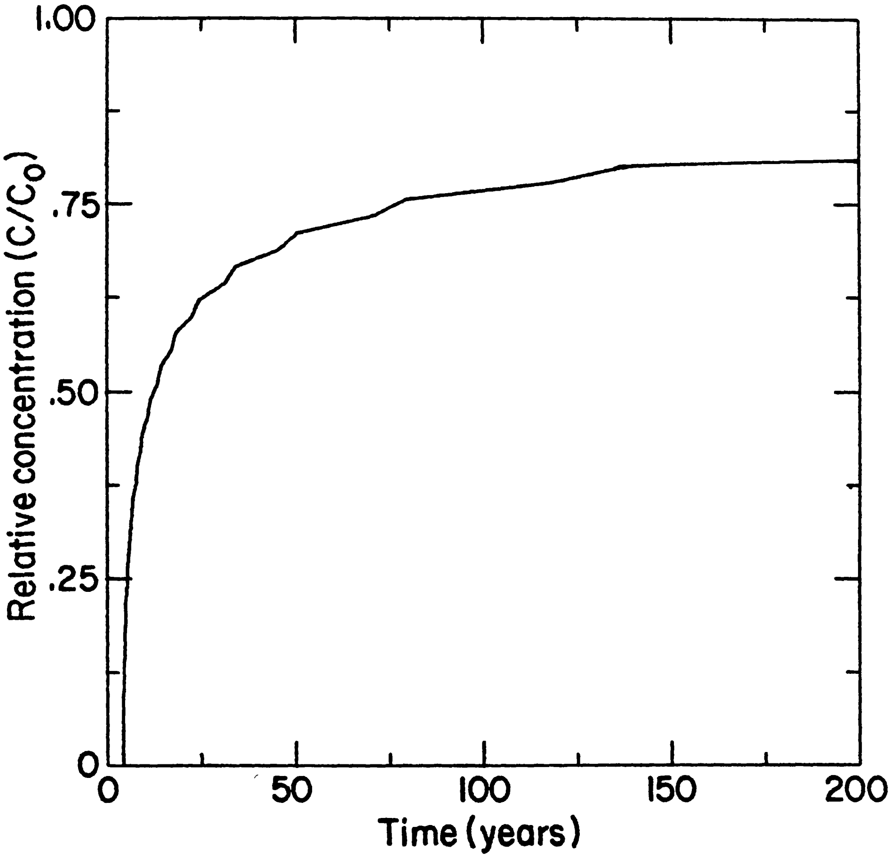


Figure 3.40. Relative Concentration-Time Plot for the RESSQ Example Discussed in the Text: the Case Without Regional Flow

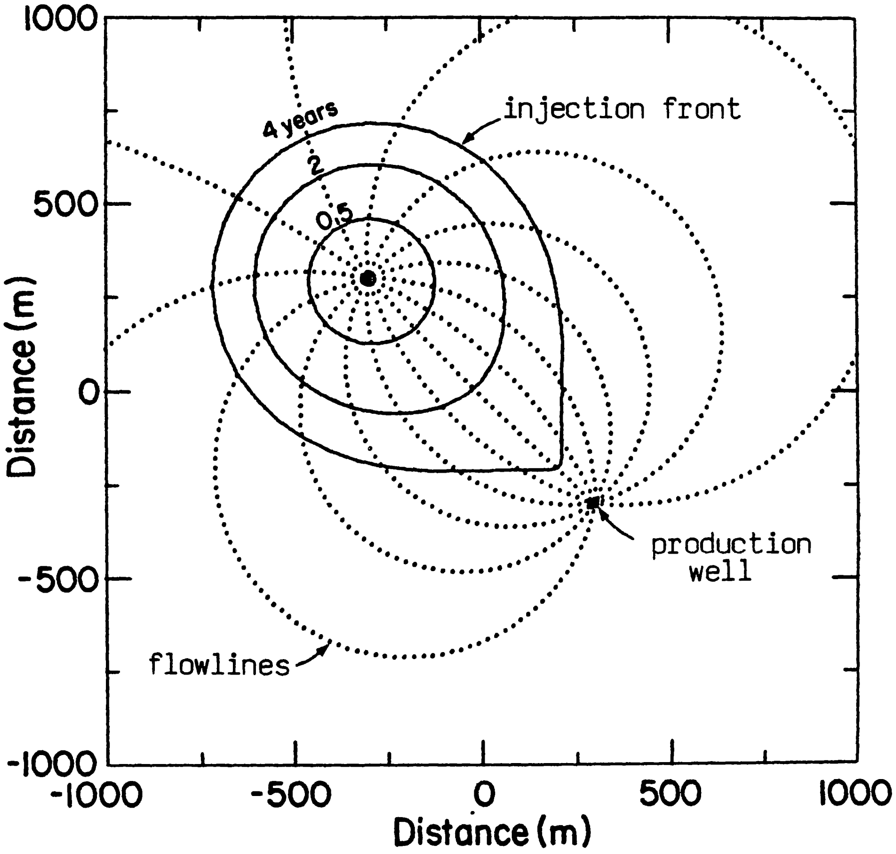


Figure 3.41. Injection Front at Select Times for the RESSQ Example Discussed in the Text: the Case Without Regional Flow

The example just discussed can be expanded by using RESSQ to incorporate the effects of combining a regional/natural flow velocity of 0.45 ft/day (50 m/yr) oriented perpendicular to a line joining the two wells (i.e., $\mathring{a} = 45$). Combining the equations for the **x-** and **y-**components of $\mathbf{V}_{\mathbf{natural}}$ and $\mathbf{V}_{\mathbf{pumping}}$ developed earlier, the **x-** and y-components of the net velocity, $\mathbf{V}_{\mathbf{net}}$, can be obtained:

$$V_{x,net} = (V_n * \cos \mathring{a})$$

$$- \Sigma_p(Q_p / 2 * \pi * h * \text{Ø}_e) * ((x - x_p) / ((x - x_p)^2 + (y - y_p)^2)$$

$$+ \Sigma_i(Q_i / 2 * \pi * h * \text{Ø}_e) * ((x - x_i) / ((x - x_i)^2 + (y - y_i)^2)$$

and

$$V_{y,net} = (V_n * \sin \mathring{a})$$

$$- \Sigma_p(Q_p / 2 * \pi * h * \text{Ø}_e) * ((y - y_p) / ((x - x_p)^2 + (y - y_p)^2)$$

$$+ \Sigma_i(Q_i / 2 * \pi * h * \text{Ø}_e) * ((y - y_i) / ((x - x_i)^2 + (y - y_i)^2)$$

For each equation, the first term represents the contribution of the regional flow velocity. The second term represents the sum of the contributions from the pumping wells (symbolized by the summation sign and the subscript '**p**'), and is a negative quantity because flow is being removed from the system. The remaining term is the sum of the contributions from the injection wells (symbolized by the summation sign and the subscript '**i**') and is a positive quantity because flow is being added to the system.

Using these equations, RESSQ generated Table 3.12, which presents the arrival times of the waste water flowing along flowlines between the injection and supply wells, depicted graphically in Figure 3.42. Note

Table 3.12. Time of Arrival of Flowlines for RESSQ Example Discussed in the Text: the Case with Regional Flow

Injection Well Flowline Number	Captured by the Production Well?	Arrival Time (years)	Departure Angle of Flowline (β)
1	no	---	0
2	no	---	8
3	no	---	16
4	no	---	24
5	no	---	32
6	no	---	40
7	no	---	48
8	no	---	56
9	no	---	64
10	no	---	72
11	no	---	80
12	no	---	88
13	no	---	96
14	no	---	104
15	no	---	112
16	no	---	120
17	no	---	128
18	no	---	136
19	no	---	144
20	no	---	152
21	no	---	160
22	no	---	168
23	no	---	176
24	no	---	184
25	no	---	192
26	yes	21.4	200
27	yes	11.6	208
28	yes	8.7	216
29	yes	7.1	224
30	yes	6.2	232
31	yes	5.5	240
32	yes	5.1	248
33	yes	4.8	256
34	yes	4.7	264
35	yes	4.6	272
36	yes	4.7	280
37	yes	4.8	288
38	yes	5.1	296
39	yes	5.5	304
40	yes	6.1	312
41	yes	7.0	320
42	yes	8.6	328
43	yes	11.5	336
44	yes	22.4	344
45	yes	200.0	352

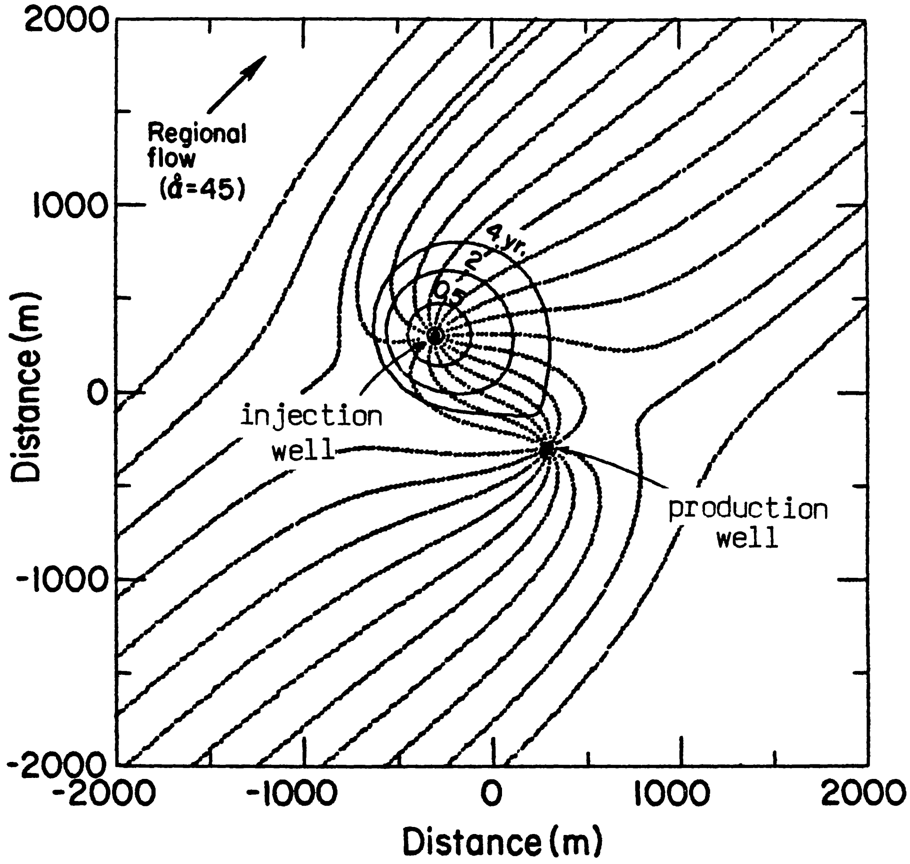


Figure 3.42. Flowline Plot for the RESSQ Example Discussed in the Text: the Case with Regional Flow

that only 20 of the 45 flowlines emanating from the injection well are now able to reach the supply well. The remaining 26 are washed away by the regional/natural flow system. This causes the maximum relative concentrations (44 percent) of waste-water contaminants in the water discharged from the water-supply well to be considerably lower (Figure 3.43) than was the case where regional flow was not considered (Figure 3.40).

The contours of the position of the waste-water front after 0.5, 2, and 4 years are also shown in Figure 3.42; it is quite evident that the regional flow has displaced the contours downgradient as compared with the earlier presentation neglecting regional flow (Figure 3.41). To some, this might tend to imply some mitigation of the urgency of the situation for the water-supply well owner; however, one should note that the injected waste-water front arrives at 4.6 years (the 35th flowline, Table 3.12). This is not so different from the 4.3 years calculated in the discussion where regional flow was neglected. Hence, in this case, the regional flow is adequate to substantially reduce the contaminant loadings to the supply well, but it does not significantly affect the arrival time of the contaminant plume, despite the fact that the injection well is a healthy distance directly across the regional gradient from the pumping well.

These few examples have been offered to illustrate the power and the simplicity of some analytical techniques, primarily velocity plots, to rapidly estimate the impact to pumping wells from nearby contaminant plumes. As indicated throughout, a wide variety of situations can be examined.

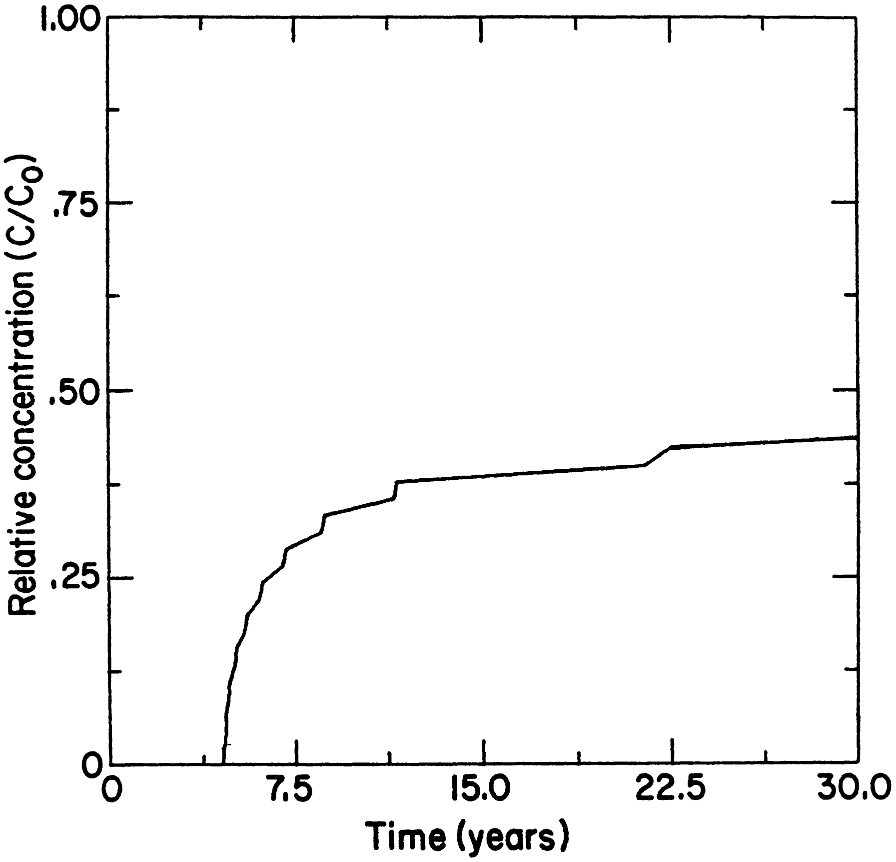


Figure 3.43. Relative Concentration-Time Plot for the RESSQ Example Discussed in the Text: the Case with Regional Flow

3.5.4 Examples for Remedial Actions

A typical use of pumping schemes is to effect hydrodynamic control over a plume, either for long-term stabilization or for withdrawal and treatment. The flowlines generated by pumping a well from an idealized aquifer (homogeneous, isotropic, constant density, etc.) under several different natural flow conditions are shown in Figure 3.44. The well is pumping 1,000 cu.m/day from a 10 m thick aquifer having a porosity of 0.10 and a hydraulic conductivity of 100 m/day has uniform radial flow under stagnant aquifer conditions (e.g. natural flow velocity equal to zero). When a mild hydraulic gradient (0.0001) is imposed on the same system (Figure 3.44b), the resulting natural flow velocity (0.1 m/day) is insufficient to significantly affect the flowlines, and uniform radial flow is nearly maintained. With a more moderate hydraulic gradient (0.001), the resulting natural flow velocity (1.0 m/day) is sufficient to sweep away many of the flowlines and the capture zone is clearly evident (Figure 3.44c). Where a steep gradient (0.01) is present, the capture zone diminishes to a small fraction of the zone of pressure influence (Figure 3.44d).

Consider extending the example illustrated by Figure 3.44. First, assume a line of five wells lies perpendicular to the direction of natural flow (Figure 3.45). Each of the five wells pumps 200 cu.m/day so that the total pumpage of the five wells is the same as that of the single well in Figure 3.44. Under stagnant aquifer conditions to low natural flow velocities (Figures 3.45a and 3.45b) there does not seem to be any difference in the effectivenss of the pumpage from the five wells as compared with the single well case (Figure 3.44a and 3.44b). The

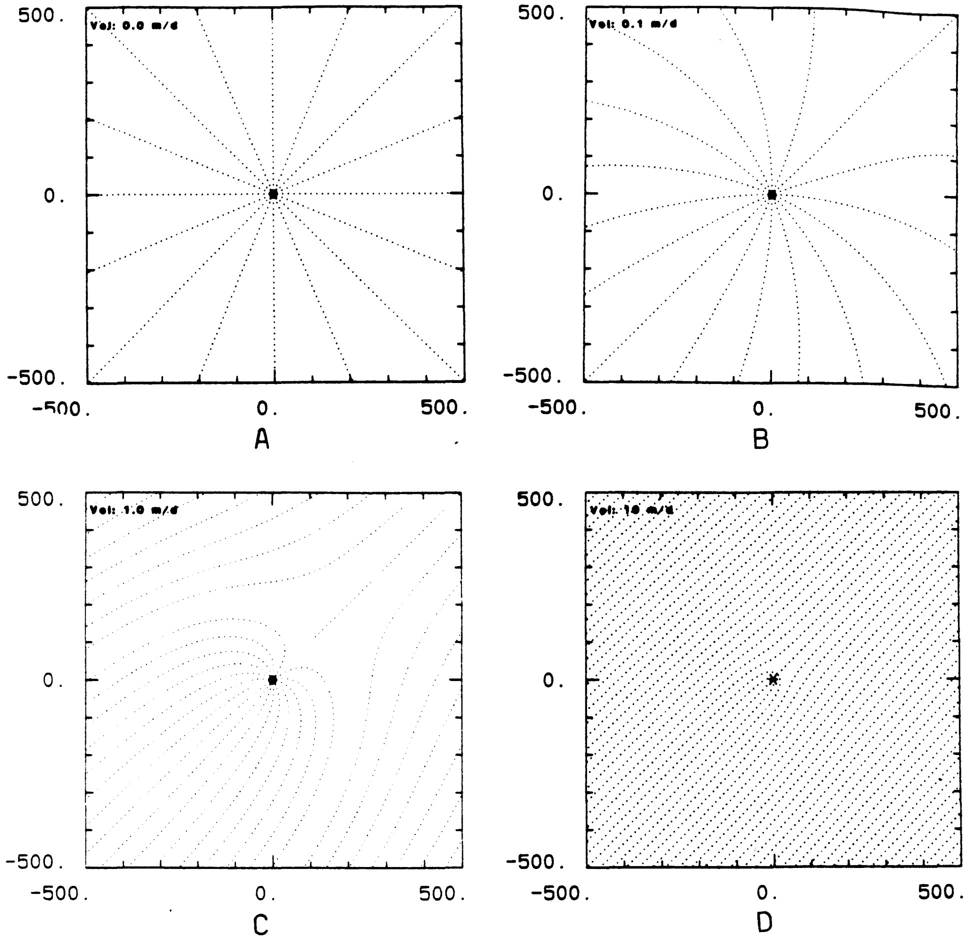


Figure 3.44. Flowline Plots for a Well Discharging 1,000 m^3/day from an Aquifer with $h = 10$ m, $K = 100$ m/day, $\phi_e = 10\%$

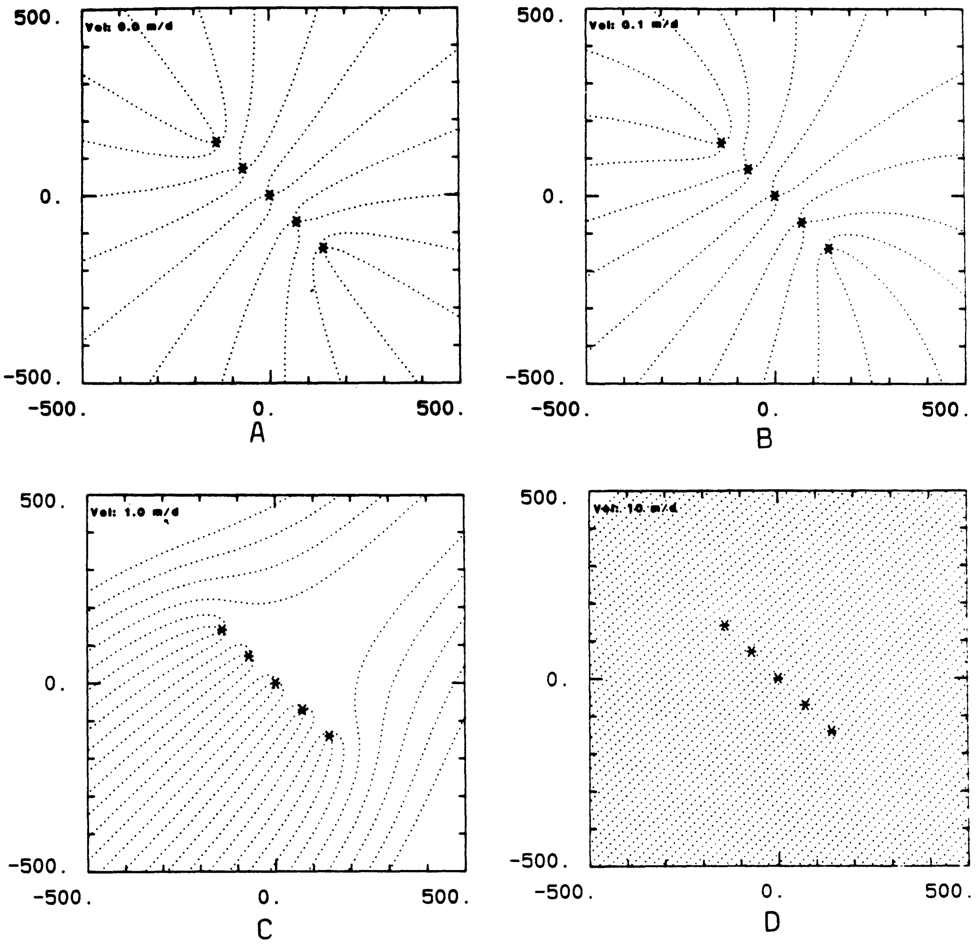


Figure 3.45. Flowline Plots for a Line of Five Wells, Discharging 200 m^3/day Each from Same Aquifer as in Figure 3.44

situation changes markedly if moderate to high natural flow velocities are present, however, as depicted in Figures 3.45c and 3.45d. As the natural flow velocity increases, the capture zone of each of the five wells diminishes to a point where adjacent capture zones no longer overlap and flowlines run on through the line of wells. By contrast, the capture zone of the single well pumping 1,000 cu.m/day does not develop holes, but does diminish in size to well below the perceived size of the leaky collective capture zone of the line of five wells.

In actuality, there is no difference between the true collective size of the capture zones generated by the five wells and that generated by the single, high-flowrate well. By rearranging the expressions for capture zone dimensions given earlier, it is possible to show that the maximum width of the capture zone of a well is directly and linearly related to its flow rate (Q), and is inversely related to the natural flow velocity:

$$r_{sp} = Q \;/\; (2 * \pi * h * \text{Ø}_e * V_{natural})$$

and

$$W_{max,cz} = 2 * \pi * r_{sp}$$

so, by substitution,

$$W_{max,cz} = Q \;/\; (2 * \pi * h * \text{Ø}_e * V_{natural})$$

For the example discussed here regarding a single well pumping 1,000 cu.m/day, the maximum width of the capture zone is 1,000 m when the natural flow velocity is 1.0 m/day, and is 100 m when the natural flow velocity is 10 m/day. Each of the five wells in the second example discussed pumps at a flow rate equal to one-fifth the flow rate of the well in the first example (200 cu.m/day), and each, therefore, has a capture zone maximum width that is one-fifth that of the single well (200 m). Hence, by comparing Figure 3.44 with Figure 3.45, it is seen that the way in which the total pumpage is distributed does directly affect the distribution of the capture zone(s), but does not affect the magnitude or total area of the capture zone(s).

Also to be seen in Figures 3.44 and 3.45 is that increasing the natural flow velocity estimate can have dramatic impact on the effectiveness of the pumping strategy. Given the order-of-magnitude uncertainty so often associated with hydraulic conductivity estimates, it is not surprising that many seemingly acceptable remedial action schemes are doomed to fail miserably.

A more complicated example provides further illustration of these points. Assume that we have the same aquifer conditions and total pumpage limitation (1,000 cu.m/day) as the preceding examples. We will distribute the pumpage uniformly by pumping each of eight wells at 125 cu.m/day. The eight wells are evenly spaced around a circle of 200 m radius. We are trying to hold a plume within the circle. With stagnant aquifer conditions to low natural flow velocities, the plume appears to be stable; no flowlines pass through the circle (Figures 3.46a and 3.46b). At moderate to high natural flow velocities, however, the situation is quite different; flowlines readily pass through the circle, indicating that the plume stabilization attempt has failed (Figures 3.46c and 3.46d).

A pump and treat scenario can be examined by modifying the example shown in Figure 3.46 to change the operation of the eight wells from pumping to injecting and by adding a major pumping well in the center of the circle. The single pumping well will withdraw 1,000 cu.m/day from the

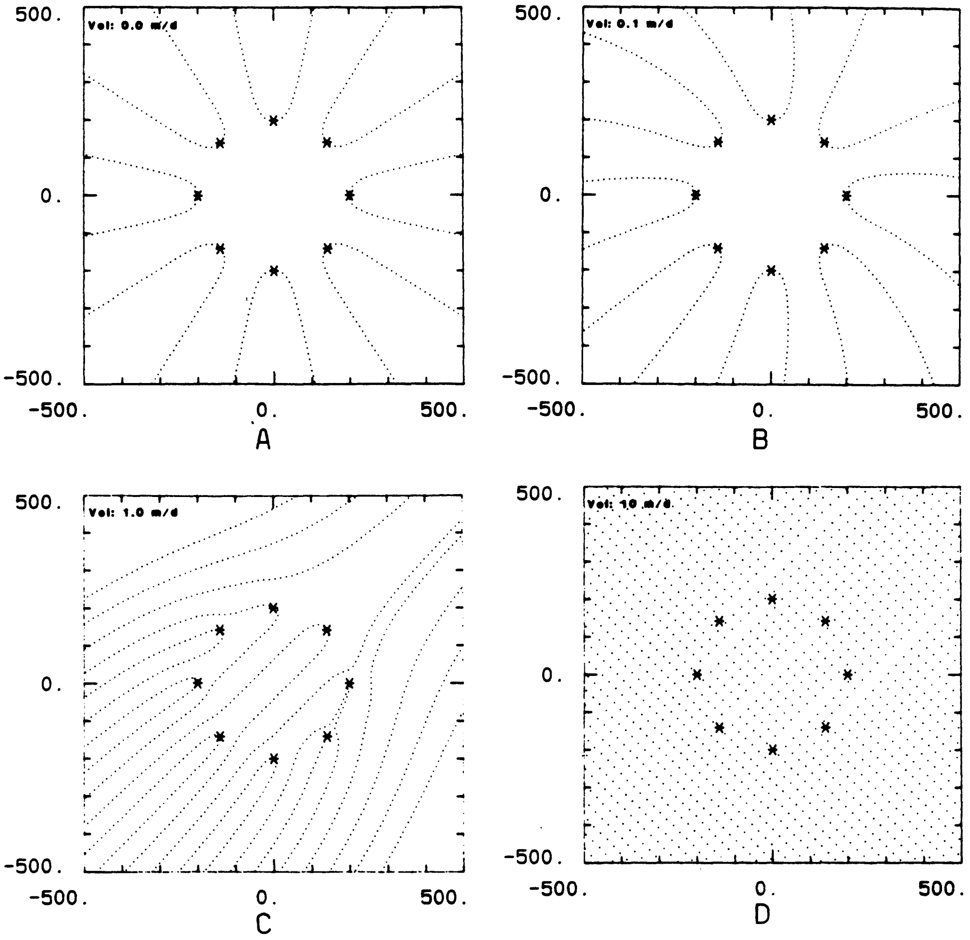


Figure 3.46. Flowline Plots for a Circle of Eight Wells, Discharging 125 m^3/day Each from Same Aquifer as in Figure 3.44

plume. The withdrawn water will be treated and re-injected into the eight injections wells at 125 cu.m/day each. At zero to low flow velocities, the injected water flows radially toward the central pumping well, forming a closed loop for recovery and treatment of plume (Figures 3.47a and 3.47b). At moderate to high natural flow velocities, the recovery loop is broken and an increasing amount of the injected water and the plume are swept away by the regional flow (Figures 3-47c and 3-47d).

It must be emphasized that the cones of impression or depression of the wells overlap significantly for all of the multiwell examples dicussed so far. Despite those overlaps, the net surface resulting from the natural gradient and the water level changes due to pumpage and/or injection is shaped such that the flowlines are truly as presented here.

It was quite clear in each of the preceding examples that the pumping strategy began to fail as the natural flow velocities became appreciable. The tendency to fail is generally becoming evident at a natural flow velocity of 1.0 m/day is beyond question at a natural flow velocity of 10 m/day. In Figure 3.48 the natural flow velocity has been reduced to 0.5 and 0.4 m/day for the last two examples, respectively. Breakthrough of the flowlines (failure of the pumping strategy) occurs somewhere between the 0.4 and 0.5m/day natural flow velocities. Similar comparisons for the first two examples are not presented because flow line breakthrough does not apply to the first example (a single production well) and the flow line did not indicate breakthrough at 1.0 m/day for the second example (a line of five wells).

The presence of an unknown well is being studied in Figure 3.49. A major pumping well (1,000 cu.m/day) has been arbitrarily located

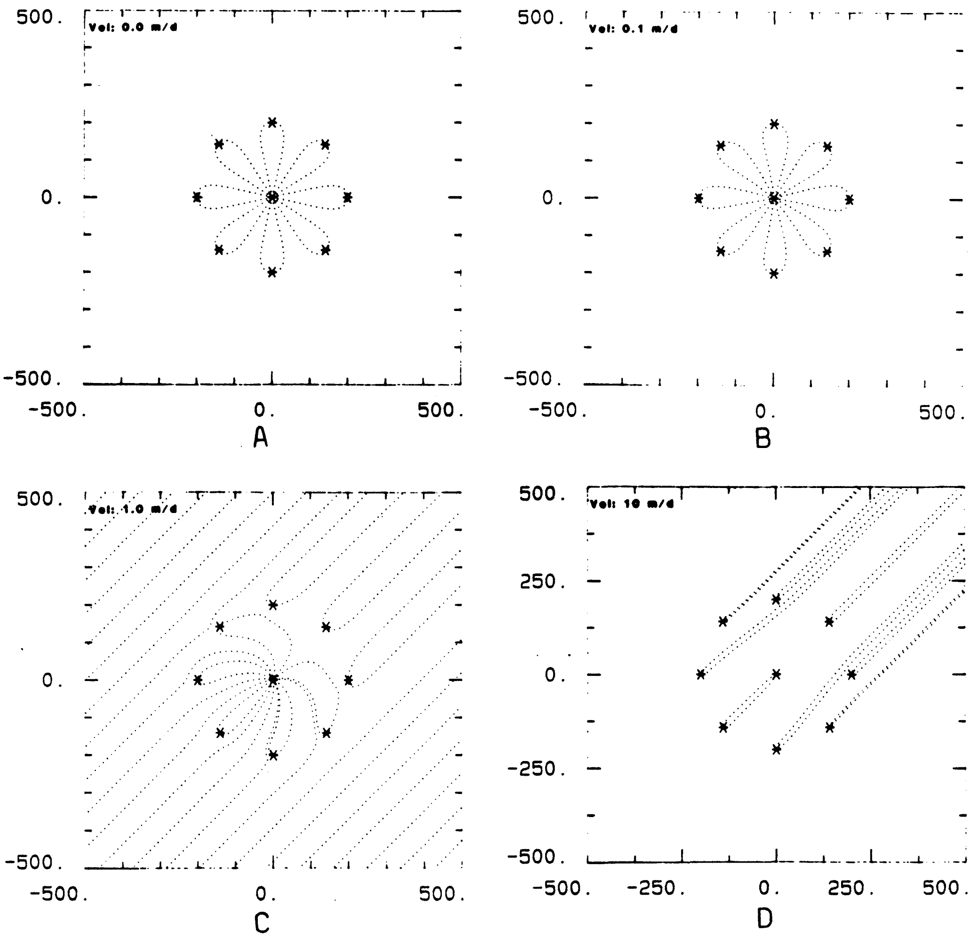


Figure 3.47. Flowline Plots for a Modification of the Figure 3.46 Example That Depicts a Plume Recovery System

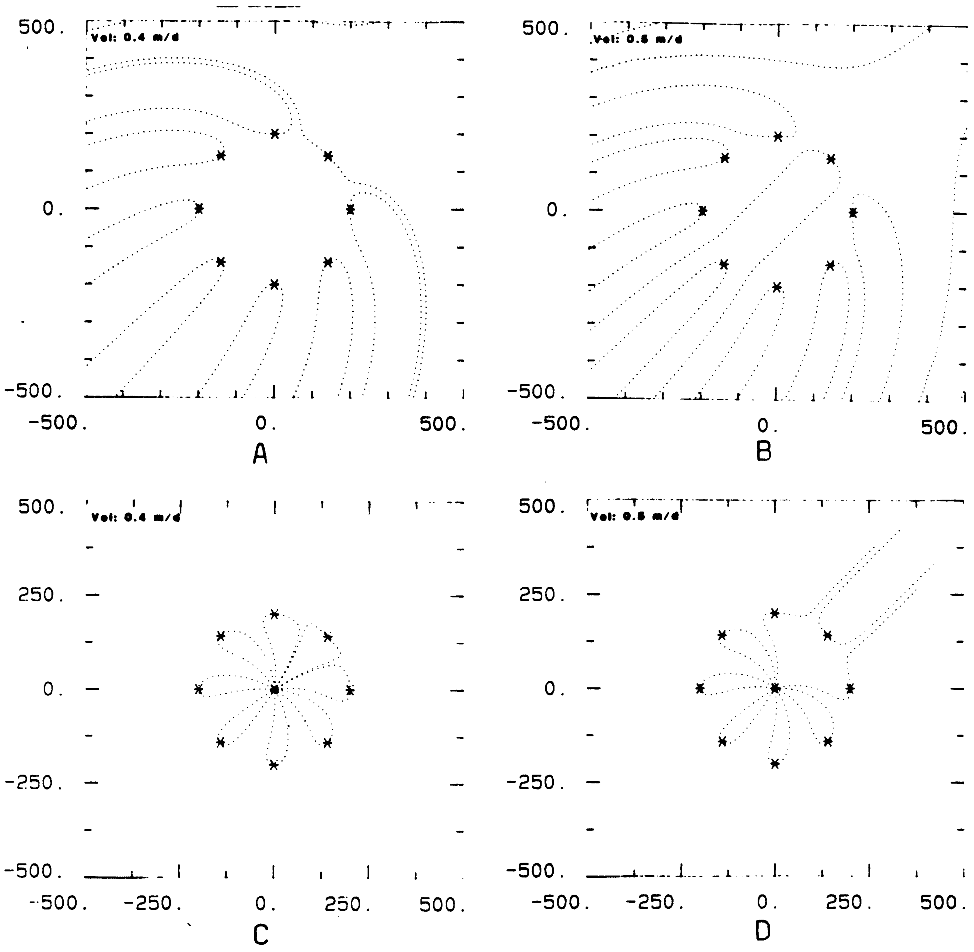


Figure 3.48. Flowline Plots Showing Breakthrough / Failure Points for the Examples in Figure 3.44 and Figure 3.45

downgradient of the same line of five wells discussed in the second example. Naturally, under stagnant aquifer conditions, the unknwon well creates a hydraulic divide by distorting the flow field, but it does not cause breakthrough of the flowline from across the line of five wells (Figure 3.49a). With a natural flow velocity of 0.5 m/day, however, flowlines do begin to break through the line of five pumping wells (Figure 3.49b). Substantial failure of the pumping scheme occurs at 1.0 m/day natural flow velocity (Figure 3.49c). Contrast the onset of breakthrough due to unknown pumpage (Figure 3.49b) with the same situation in the absence of the unkown pumpage (Figure 3.49d). The impact of the unknown well is staggering, not only because flowline breakthroughs are occurring, but because the collective size of the capture zones of the five pumping wells is being substantially reduced.

Another illustration of the impact of an unknown well on the effectiveness of a pumping scheme is shown in Figure 3.50, which is the same example as discussed earlier (Figure 3.47) for a closed-loop aquifer rehabilitation system. Under stagnant aquifer conditions, the unknown well diverts flow away from two of the injection wells (Figure 3.50a). At 1.0 m/day natural flow velocity, the unknown well diverts flow from five of the eight injection wells (Figure 3.50b). It also allows flow to break away from the well field entirely, as indicated by the flowline leaving the uppermost injection well and heading downgradient in Figure 3.50b. The regional flowlines were omitted from Figure 3.50 and some of the diagrams in previous figures because inclusion of those flowlines would create confusion due to the excessive number of plotted points.

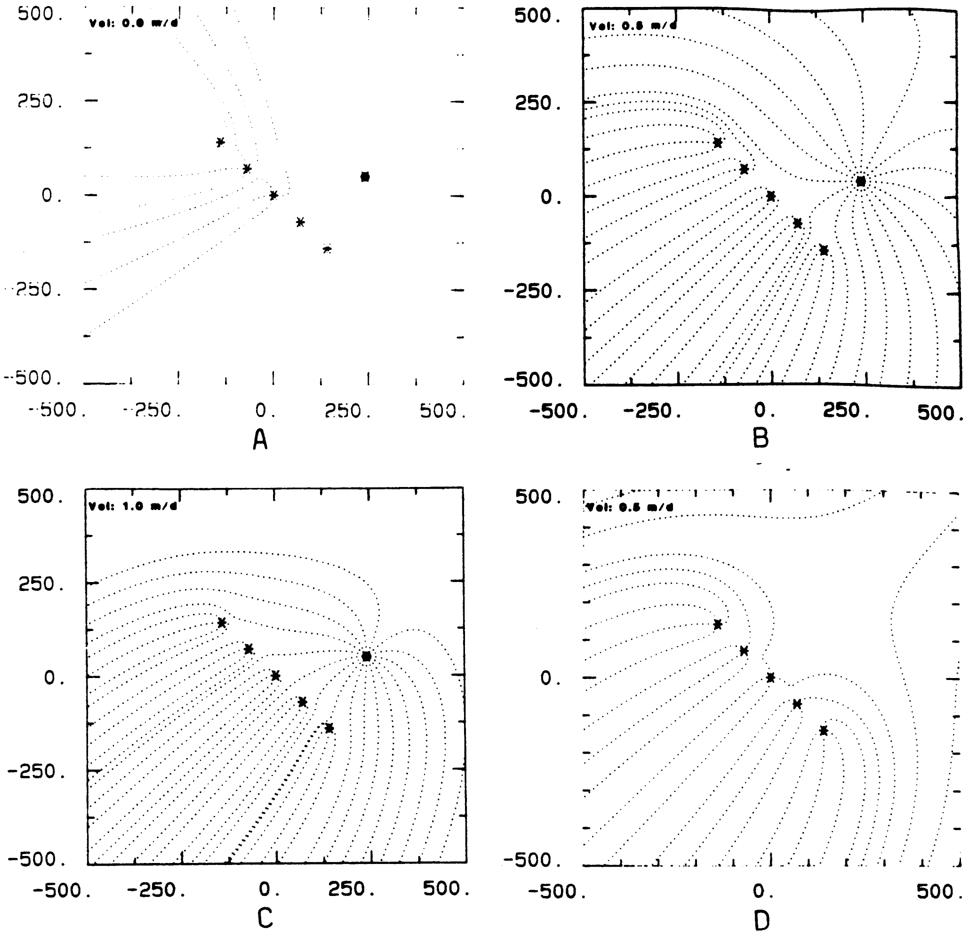


Figure 3.49. Flowline Plots Showing Potential Impact of Presence of Unknown Well on Effectiveness of Hydraulic Barrier

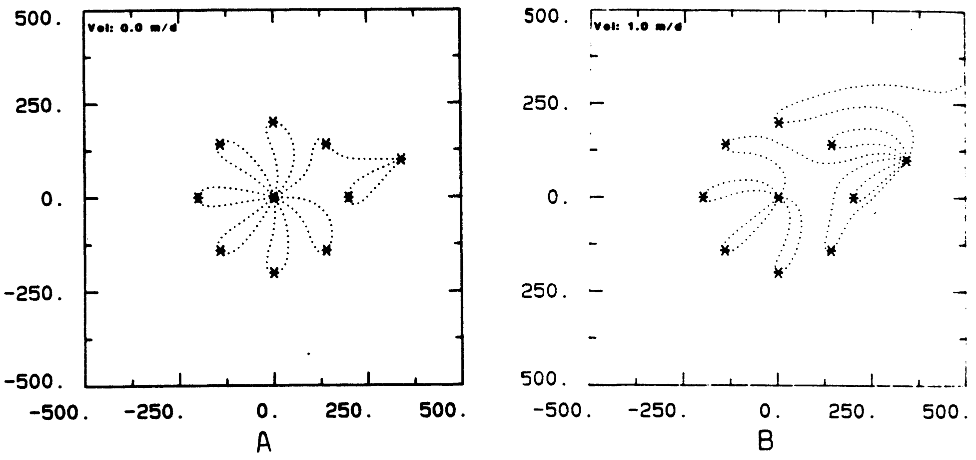


Figure 3.50. Flowline Plots Showing Potential Impact of Presence of Unknown Well on Effectiveness of Recovery System

3.3.5 Models in Perspective

Heterogeneity, anisotropy, partial penetration and other factors distort drawdown patterns and associated velocity distributions. If known, such influences can be used to enhance recovery efficiencies for remedial actions. If unknown, such influences may cause recovery efficiencies to be substantially lowered. Similarly, predictions of plume migration in non-ideal aquifers under non-pumping/natural flow conditions will be strengthened by specific knowledge regarding the occurences, extent and magnitude of the non-ideal condition(s). Such predictions may be seriously in error if non-ideal conditions are not evaluated properly.

Denial of property access, loss by vandalism and unpredictable operation of nearby wells are also major sources of uncertainty in predicting contaminant migration and in designing remedial actions. Though commonly perceived to be less of an impact on optimizing pumping strategies than non-ideal aquifer conditions, these factors may indeed be the most uncontrollable and the most detrimental to operational success. Other factors that have major impacts are the physiochemical attenuation and biotransformation potentials of the individual contamiant; however, it is not yet economically feasible to conduct adequately detailed studies of these potentials on a routine site-specific basis. Finally, a factor often overlooked that greatly impacts optimization efforts is the risk of mechanical and electrial operational failure; adequate contingency plans must provide certain minimal levels of excess/reserve capacity and redundancy of key system components.

The capture zones of wells do not equal their associated zones of pressure influence (cones of depression), except for stagnant aquifer conditions. Velocity distribution plots must be constructed to define potentials of contaminant migration. In particular, plotting the flowlines for various scenarios involving pumping and/or injection wells subject to a specific natural flow velocity can greatly assist the ground-water professional in selection of an optimal pumping strategy.

CHAPTER IV

CASE HISTORIES

4.1 Experience as the Ultimate Assessment Guide

Case histories are presented here to illustrate some of the technical, economic, and managerial problems that occur when faulty data or incorrect conceptual models are generated in the course of ground-water contamination assessments. The magnitudes of errors associated with such problems are less difficult to appreciate when real life situations are reviewed than when perusing theoretical treatises. Intuitions so derived can be used to guide applications of mathematical models to specific problems so that better remedial actions can be recommended to decisionmakers. To set the stage for this, it is useful to recap the generic precautions that should be observed when anticipating the use of mathematical models that will be used to quantify ground-water contamination assessments.

4.1.1 Stereotypical Applications of Models

As stated in preceeding chapters, mathematical models are simplifications of reality that may or may not faithfully simulate the actual situation. Typically, attempts are made to mimic the effects of hydrogeologic, chemical, and biological processes in practical applications of models. These almost always involve idealizations of known or suspected features of the problem on hand. For example, the stratification of alluvial, fluvial, and glacial deposits may be assumed to occur in uniformly thick layers, despite the great variability of stratum thicknesses found in actual settings. Large blocks of each stratum are usually assumed to be homogenous. Sources of chemical input are commonly assumed to have released contaminants at constant rates over the seasons and years of operational changes that the sources were active. The areal distribution of rainfall and the actual schedules of pumpage from production wells are also artificially homogenized in most mathematical modeling exercises.

These idealizations are often made necessary by a lack of the appropriate historical records and a scarcity of field-derived parameter estimates; all reduce the reliability of predictions made with models. The degree of usefulness of a model is therefore directly dependent on the subjective judgements that must be made in data collection and preparation efforts, prior to attempting mathematical simulations. This is true not only in a quantitative sense, but also in a qualitative sense because it is the data gathering phase of a project that begets the conceptualization on which the model will be based.

4.1.2 Real-World Applications

To illustrate these points, the highlights of four very different contamination problems will be described. The first case history describes how the method of drilling during investigations at a chrome plating operation might have caused cross-contamination from a severely contaminated shallow aquifer to a deeper domestic water-supply aquifer.

The second involves a relatively limited contamination incident arising from a very small source that is located downgradient of two major public water-supply wells; well hydraulics, chemical time-series sampling, and velocity distributions were key to its solution. The third case briefly discusses solution density effects on a plume from a brine disposal lagoon. Finally, a complex situation at a chemical reprocessing facility is used to signify the need for three-dimensional flow, sorption, and biotransformation considerations.

The common theme that is shared by the four case histories is one that should apply everywhere; that of seeking to define the relative influences of natural processes affecting contaminant transport in order to optimize the assessment and remediation of the problem. It is the establishment of a valid conceptual model of what is happening at these sites that is most important, not the application of a particular mathematical model.

Case History No.1

4.2.1 Background and Setting

Between 1956 and 1975, process wastes had been discharged into a dry-well in an unlined disposal pit at a hard-chrome plating facility (Figure 4.1). A review of well logs in the general area and limited investigations of the site showed that it is underlain by unconsolidated Quaternary alluvial deposits of clay, silt, sand and gravel. Two aquifers are encountered within 45 feet of the ground surface. The upper aquifer is an unconfined clayey silt stratum that is only slightly permeable (K_{horiz} = 0.6 to 2.2 feet/day) and varies in thickness from 15

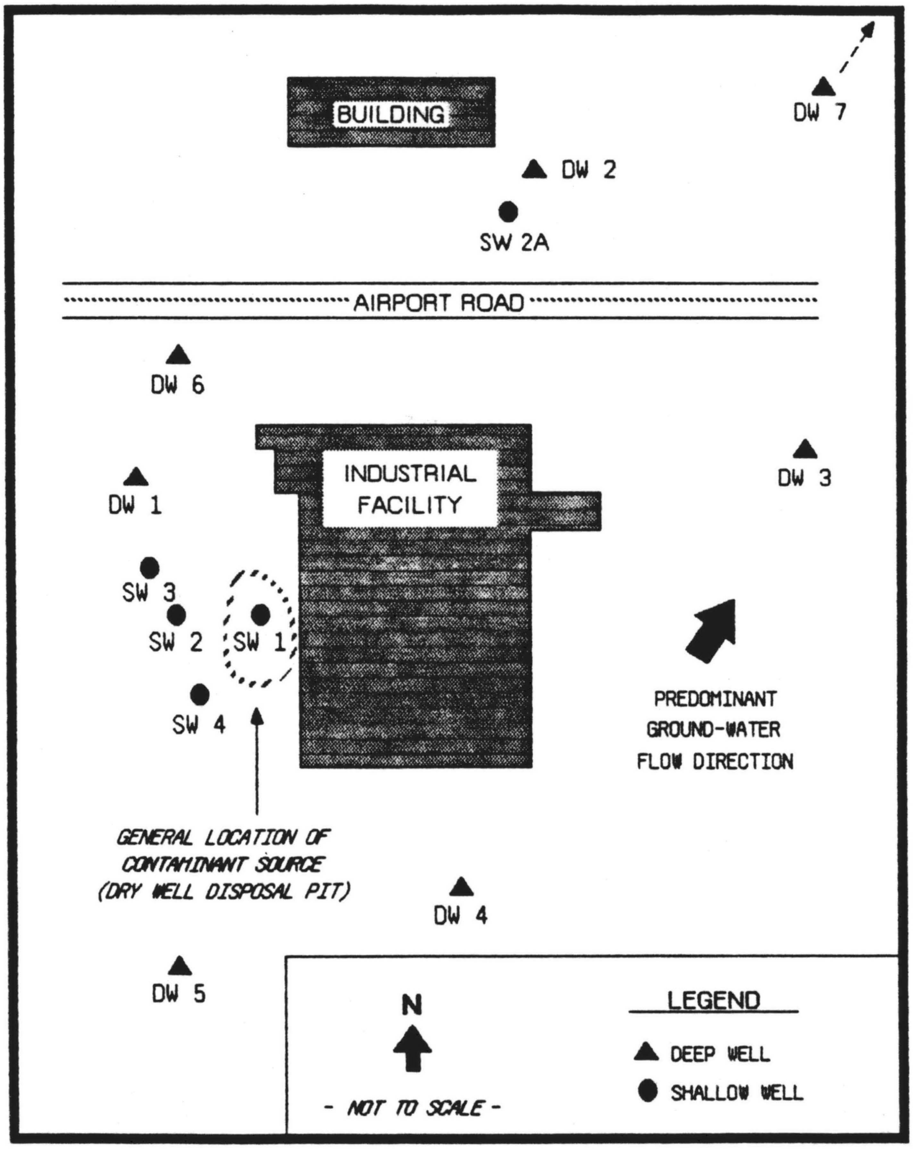


Figure 4.1. Monitoring Well Location Map for Case History No.1

to 20 feet (Ecology and Environment, 1985). The lower aquifer also averages 15 to 20 feet in thickness, but is a highly permeable confined sand and gravel stratum (K_{horiz} = 18 to 22 feet/day).

The two aquifers are separated by a relatively impermeable silty clay stratum (K_{vert} = 0.001 to 0.01 feet/day), that ranges in thickness from five to ten feet beneath the site, but that is known to be irregular and discontinuous over the broader local area. The upper aquifer is not used as a domestic water supply, but the lower aquifer is being used by local residents. Concerns expressed by local and state officials about potential ground-water contamination led to the installation of several ground-water monitoring wells to assess water quality beneath the site (Figure 4.1).

4.2.2 Drilling Technique

Since local hydrogeologic conditions were not known in detail, and because potential cross-contamination of the underlying production aquifer was of concern, it was initially proposed to use a conservative approach for well installations. The monitoring wells tapping the lower aquifer were to be drilled with a cable tool drill rig to the base of the silty clay aquitard. A protective steel casing would be installed and cemented in place. The bottom of the borehole would be sealed with grout during this phase, and then drilling of a smaller diameter well would commence through the grout to the underlying aquifer (Figure 4.2).

When time and cost estimates were prepared for this approach, it was rejected by management as too slow and expensive. Technical arguments that supported the management decision said hollow-stem augering could

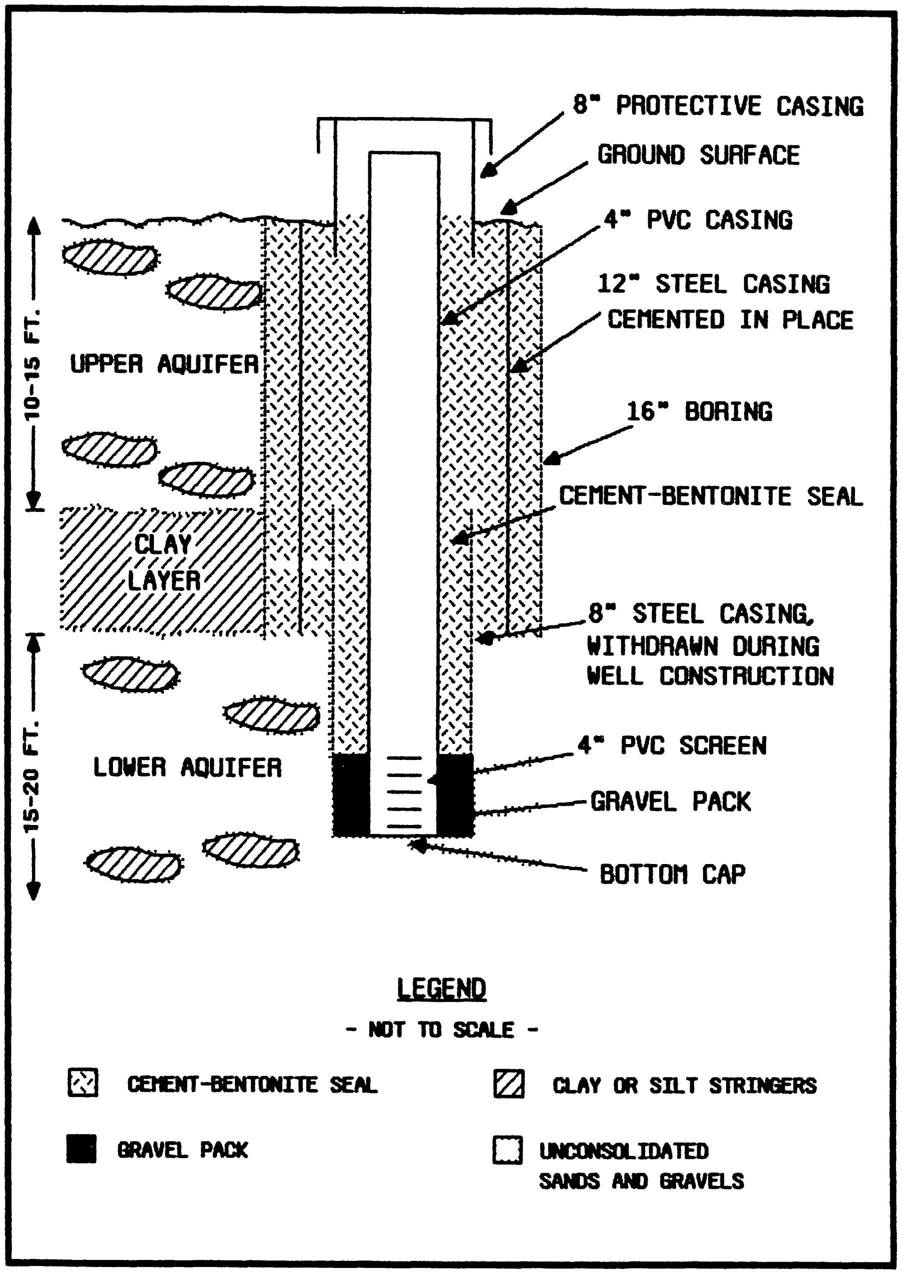


Figure 4.2. Monitoring Well Installation Design to Ensure Against Cross-Contamination of Each Stratum Encountered

accomplish the desired results if the wells are properly grouted. The potential for cross-contamination by movement of sediments or fluids from one stratum to another during the augering process was considered to be insignificant. The monitoring wells were, therefore, installed with an eight-inch diameter hollow-stem auger. After the borings had been augered to the required depth, a two-inch diameter PVC casing with ten-foot length of screen was installed, gravel packed, and the annular space was filled with bentonite-cement grout.

4.2.3 Initial Chemical Samplings

Nine monitoring wells were installed and sampled during the summer of 1983; four in the upper aquifer, and five in the lower aquifer. All the monitoring wells were sampled two weeks after installation (Figure 4.3), and re-sampled in January, 1984 (Figure 4.4). High concentrations of chromium were found in all the shallow monitoring wells, indicating that the upper aquifer was grossly contaminated. Shallow well SW-1, which is closest to the suspected source, showed the highest concentration. The adjacent deeper well, DW-1, had only background levels of chromium, indicating that contamination had not spread to the lower aquifer. Results from a subsequent sampling, however, showed marked increases in the chromium concentrations in well DW-1; from background levels immediately after installation, to more than 19,000 ug/l in January,1984.

During the 1983 post-installation sampling, only one of the deeper wells had shown any contamination. Well DW-2 had 3,037 ug/l of chromium, and this level rose to 9,360 ug/l in January,1984. Consequently, a shallow well, SW-2A, was drilled adjacent to well DW-2 in February, 1984

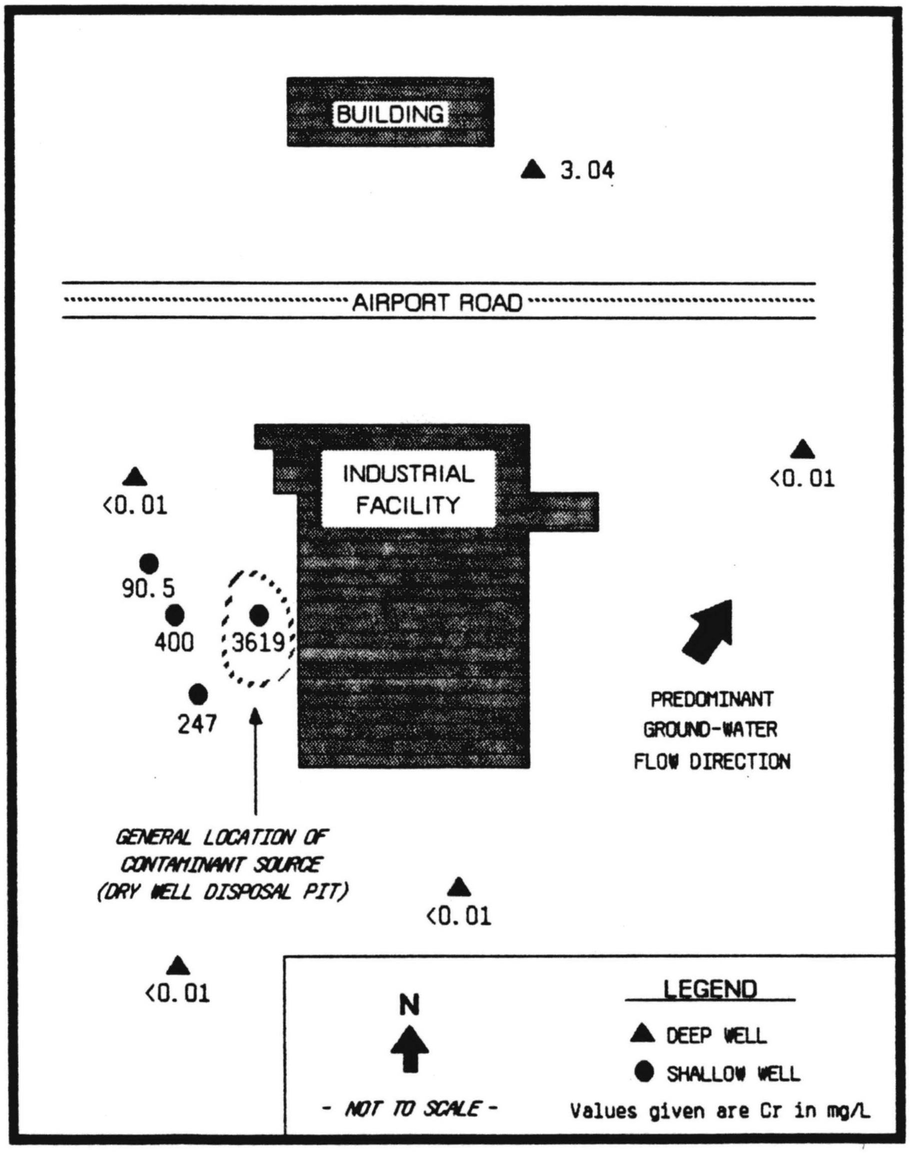


Figure 4.3. Chromium Concentrations in July, 1983 Samples

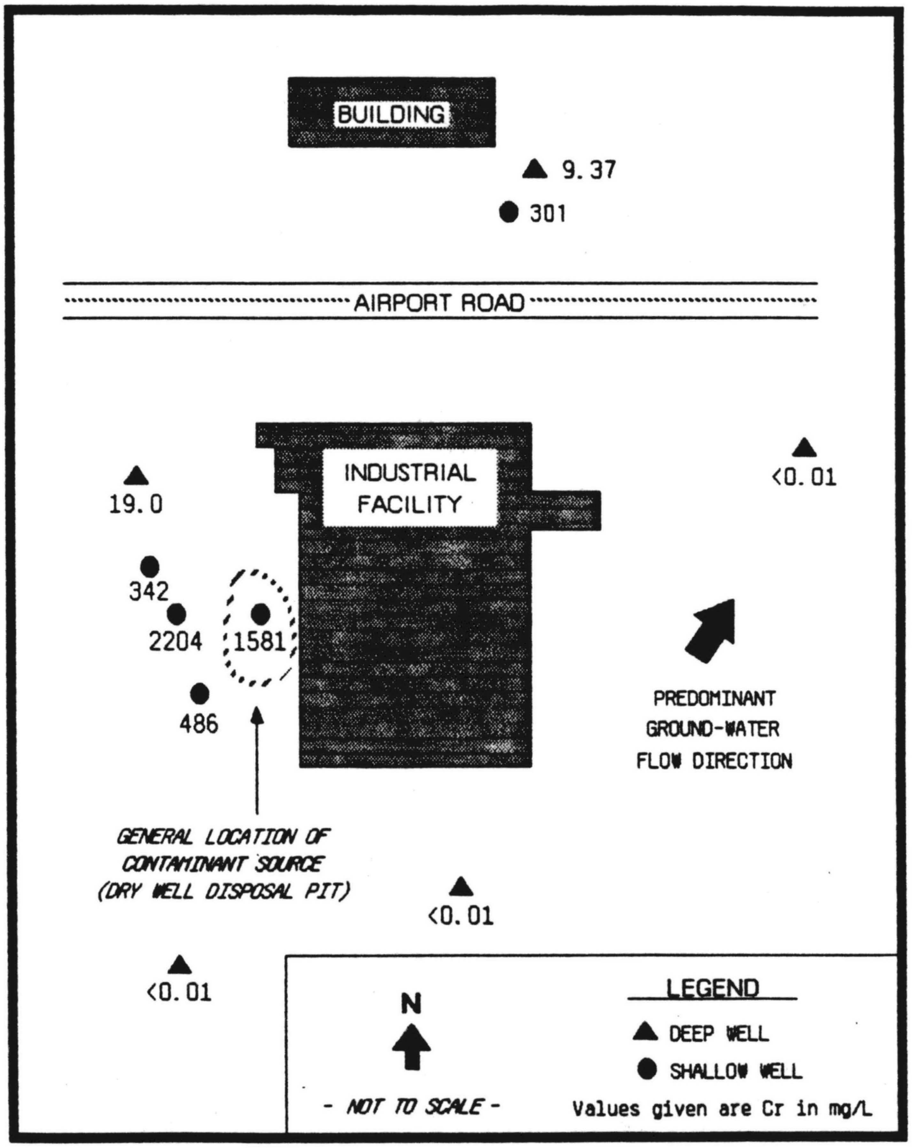


Figure 4.4. Chromium Concentrations in January, 1984 Samples

to determine the magnitude of contamination in the overlying aquifer. It was contaminated with 301,900 ug/l of chromium, confirming suspicions of extensive contamination of the shallow aquifer. The remaining deep monitoring wells, DW-3, DW-4 and DW-5 had non-detectable levels of chromium in the July,1983 and January,1984 samplings. Hence, while the shallow aquifer was grossly contaminated, the deeper aquifer seemed to be affected at only a few locations.

4.2.4 Hypotheses Advanced

These data indicated that there might be limited leakage from the upper aquifer to the lower aquifer. The possibility that the clay aquitard may be absent near deep wells DW-1 and DW-2 was discounted because substantial thicknesses of that clay stratum were encountered during drilling those wells. The piezometric heads of the two aquifers vary considerably due to local pumpage and recharge; however, the net result is an average downward vertical gradient of 0.40 (Ecology and Environment, 1985). If natural recharge of the lower aquifer has occurred, by leakage of water from the upper aquifer through the intervening aquitard, then that would explain the findings of contamination in the lower aquifer. In the absence of highly permeable discontinuities in the clay aquitard, the contaminants would have required approximately 10 years to migrate through it, if advection (0.004 feet/day in the aquitard) were assumed to be the sole transport mechanism. That is certainly within the time period of operational and post operational activity at the site (1956-present day).

The difficulty with this hypothesis is that the areal distribution of contamination in the lower aquifer is much more limited than in the upper aquifer, yet it is in a more mobile flow regime. Average horizontal flow velocities are 0.025 feet/day in the upper aquifer and 0.12 feet/day in the lower aquifer (Ecology and Environment, 1985). These field determined estimates suggest that, after migration through the clay aquitard, contaminated waters will move within the lower aquifer at a rate that is twenty times that of water movement in the upper aquifer. It is not unreasonable to assume that the entire aquitard would eventually become saturated with contaminated water under such a scenario. The aquitard would then act as a very long-term source of contaminants. In field situations that approximate that behavior, one would expect to find widespread low-level contamination of the deeper aquifer. Given the tremendous chemical loading on the aquitard in the example discussed here, the low number of contaminated deep wells is not consistent with bulk movement of contamination from the upper to the lower aquifer.

Another possible explanation for the pattern of contaminant increases observed in the deep wells is that localized leakage may have occurred as a result of passage through the disturbed zone around wellbores that penetrated to the lower aquifer. The fact confounding this interpretation is that leakage around well DW-2 would have had to occur immediately after well installation, while contamination at well DW-1 was not detected until six months later. One explanation is that fluids were only temporarily restrained from moving from the upper to the lower aquifer by the seal around well DW-1, and that is lost its effectiveness as the grout cured longer or as parts of the formation sloughed away in settlement. Leakage of fluids from the upper to the lower aquifer is not absolutely necessary to produce the observed data, however; solids from the upper aquifer may have fallen down the annular space during augering, creating a highly localized 'source'.

4.2.5 Monitoring Well Rehabilitations

Based on these observations it was decided to rehabilitate the poorly constructed wells to determine whether cross-contamination had occurred, and to eliminate the potential for further contamination of the lower aquifer. The field technique chosen to satisfy these objectives was to overdrill the affected deep wells with a large diameter cable-tool rig, so as to contain the disturbed zone around the wellbore that was possibly created by the auger drilling technique. The plan outlined in the new drilling specifications was to center the cable-tool rig on the well, pull out the surface steel casing, drill out the PVC monitoring well, and then drill an enlarged borehole for installation of a new monitoring well.

While pulling out the surface steel casing, however, the PVC well was readily withdrawn too. There was little or no evidence of grout on the surface of the PVC casing, indicating the ineffectiveness of the cement-bentonite grout emplaced with the hollow-stem auger. That observation strengthened the argument that the wells were improperly constructed; the poor grouting implies the presence of a contamination conduit.

A cable tool drill rig was re-centered on the wellbore after the withdrawal of the existing casings, and a 16-inch diameter hole was drilled to the base of the clay aquitard (about 19 feet below ground surface). A permanent 12-inch diameter steel casing was installed inside the 16-inch diameter temporary casing. As the temporary casing was withdrawn the annular space around the permanent casing was pressure-grouted with a mixture of cement and bentonite, which was allowed to cure undisturbed for 48 hours. The bottom two feet inside the steel casing was also plugged with cement-bentonite grout to prevent possible seepage from the underlying zone.

This process was followed by drilling out the cement plug and continuing to advance the borehole while driving a temporary eight-inch diameter steel casing with a drive shoe at the lower end. Drilling and driving this temporary casing continued to the original depth of the monitoring wells. A four-inch diameter PVC casing with a ten-foot length of slotted screen attached to the lower end was installed inside the temporary casing. As the temporary casing was withdrawn, the PVC well screen was gravel packed and remainder of the borehole was pressure grouted with a cement-bentonite mixture (Figure 4.2).

4.2.6 Well Rehabilitation Effects

Ground-water samples from most monitoring wells, including the two rehabilitated wells, were obtained in December,1984; the results are shown in Figure 4.5. The chromium level at deep well DW-1 dropped to less than one percent of the level reported in the January,1984 survey. The chromium level at deep well DW-2 was found to be about 30 percent less than the value reported in the January,1984 survey (Ecology and Environment, 1985). These findings generally support the

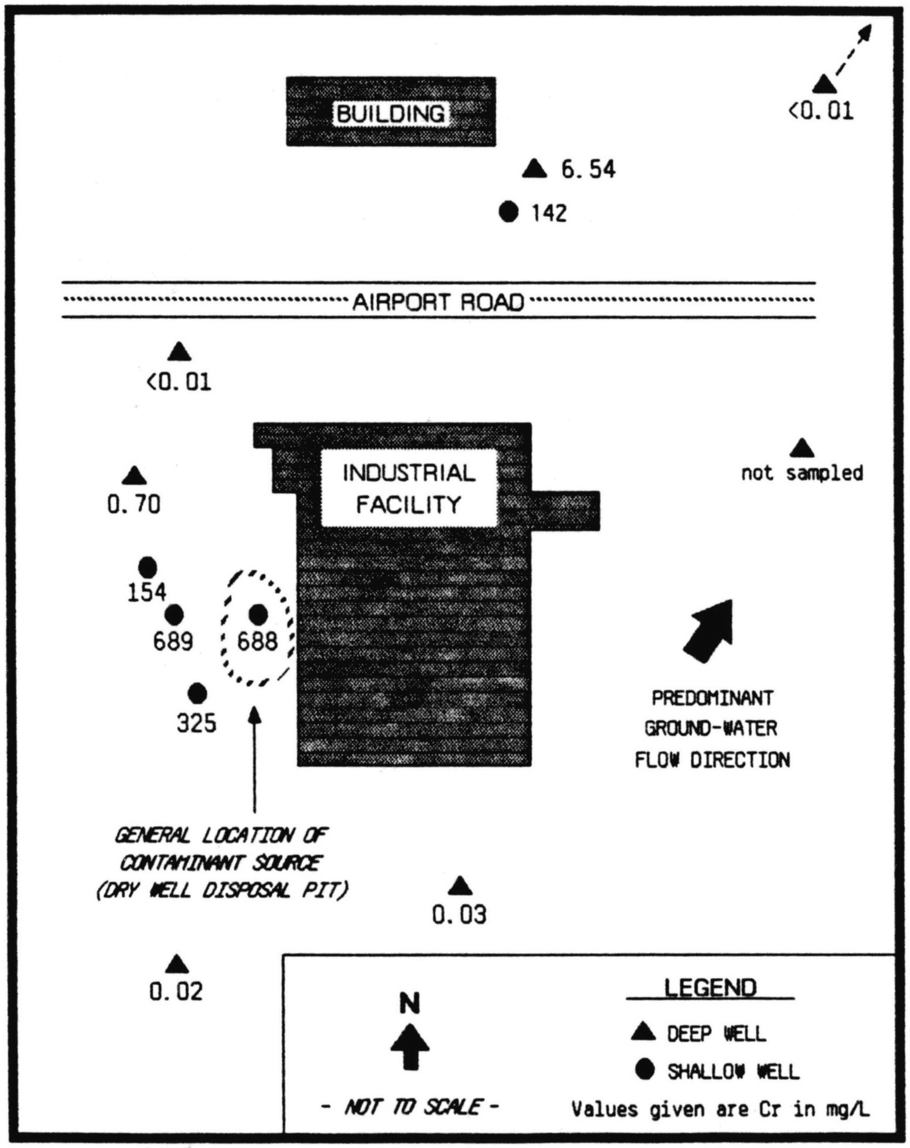


Figure 4.5. Chromium Concentrations in December, 1984 Samples

cross-contamination-by-augering hypothesis, though the minimal improvement after rehabilitation at well DW-2 is less than convincing. Additional support, however, may be found by examination of chromium levels reported for two new deep wells, DW-6 and DW-7. These wells were installed at locations downgradient of wells DW-1 and DW-2, respectively, and had no detectable levels of chromium (Figure 4.5).

As fate would have it, no survey yields wholly unambiguous data; traces of chromium were found for the first time in two upgradient deep wells, DW-4 and DW-5 (Figure 4.5). Just what these traces mean in terms of possible contamination routes, or sampling and analysis errors, is unknown at this point. As a result, complete evaluation of the necessity and effectiveness of rehabilitation of deep wells DW-1 and DW-2 will have to await additional well installations and samplings.

4.3 Case History No.2

4.3.1 Impetus for the Study

As an outgrowth of emerging concern over organic chemicals released to surface water by industrial municipal discharges, the Natural Resource Defense Council brought suit against the U.S.EPA administration (NRDC vs. Train; 8 ERC2120; D.D.C. 1976). The consent decree reached in settlement of the suit prompted U.S.EPA to issue a list defining 114 organic chemicals, 13 heavy metals, asbestos and cyanide as 'priority pollutants' to be regulated under effluent guidelines because of their toxicity to humans and aquatic life, and known or suspected carcinogenic, teratogenic or mutagenic properties. In turn, this list of 129 priority pollutants spilled over into other programs administered by the EPA, particularly drinking water and hazardous waste programs. [As a side note, given the tacit adoption of industrial effluent sampling procedures for all of the Agency's "priority pollutant" samples that has occurred in recent years, the interested reader may wish to peruse an excellent historical perspective and summary of priority pollutant sample collection and handling procedures by Banter and others (1981)].

By late 1978, the first National Organics Reconnaissance Survey (NORS) was set in motion by EPA's Office of Drinking Water to evaluate the prevalence of organic contaminants in public drinking water supplies. A follow-up of NORS in 1980 focused exclusively on ground water supplies. One of the water-supply systems randomly selected for inclusion in the 1980 follow-up study was the Lakewood Water District in Washington State. Based on grab samples, water discharged from Lakewood Water District's Well H-1 (Figure 4.6) were found to be contaminated with 10 ug/L trichloroethylene, 18 ug/L tetrachloroethylene and 61 ug/L 1,2 trans-dichloroethylene (EPA Region 10, 1980). Given that the EPA Assistant Administrator for Drinking Water had just issued Suggested No Adverse Response Levels (SNARLs, toxicologically-based health advisories) regarding the acute (10-day) and chronic (70-year lifetime) exposure limits to trichloroethylene and tetrachloroethylene due to their deleterious effects on liver and kidney cellular functions (Kimm, 1980a and 1980b), the EPA Region 10 Drinking Water Programs Branch initiated a detailed study to ascertain the potential threat to Lakewood Water District consumers.

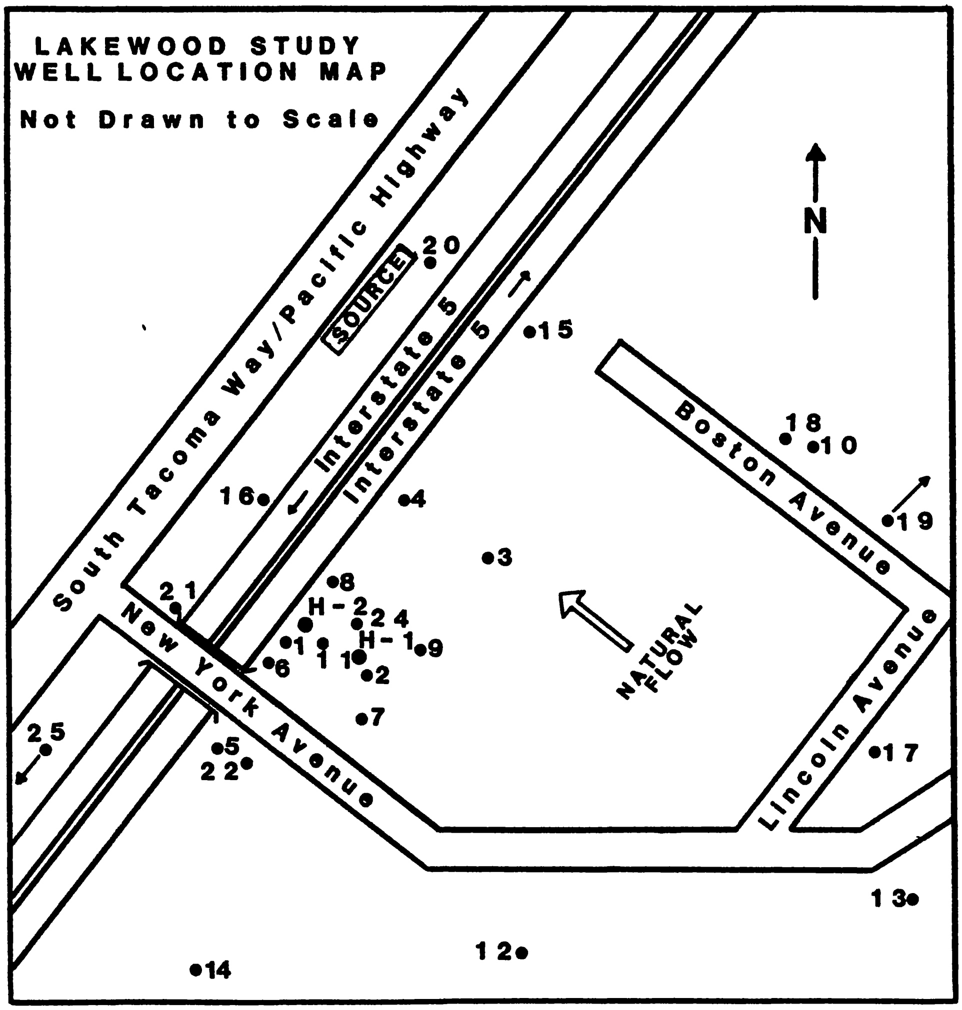


Figure 4.6. Monitoring Well Location Map for Case History No.2

4.3.2 Possible Source Mechanisms

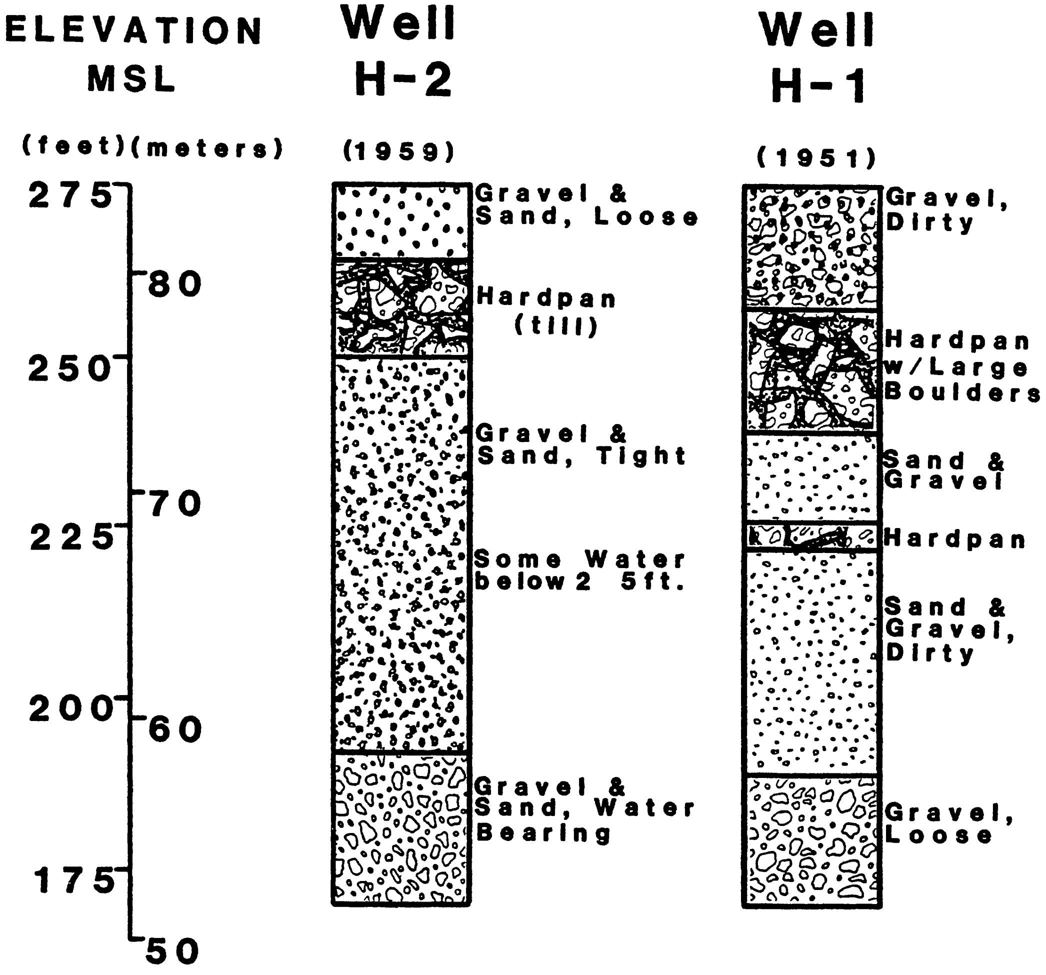
Based on discussions with Wayne Dunbar, manager of the Lakewood Water District, the author postulated that the source of the contaminants might be inside Well H-1, because it had undergone maintenance just prior to the 1980 sampling. If cleaning solvents had been used to assist in removing excess oils and lubricants from the pump column prior to its re-introduction to the well, then residues might persist and explain the contaminant observations. In part, this line of reasoning was spurred on by the recent publication of an article alerting investigators to such possible contaminant sources (Pettyjohn and others, 1981). The hypothesis seemed plausible enough, but how should it be tested?

Hydrologic and chemical intuition, as well as preliminary calculations of the impact of casing water (following the method presented by Papadopulos and Cooper, 1967), indicated that concentrations of contaminants in water discharged from Well H-1 should rapidly diminish with time from the onset of pumpage. While the well was inactive, solvent residues would bleed into water standing in the well casing and slowly attain some maximum equilibrium concentration(s). When the well was pumped, the casing water would be rapidly voided and soon the discharging water would be comprised solely of waters that had been residing in the production zone, wholly external to the well. A plot of organic chemical concentrations vs. time would thus appear to be a rapidly decaying curve. The most efficient data-gathering scheme, then, should focus on collection of several samples just after start-up of Well H-1, with a decreased frequency of sampling as pumpage continued.

The similarity of the proposed sampling scheme with that of conventional pumping tests was noted, so a suggestion was made that water-level measurements be taken concurrently at a companion well, Well H-2, located 80 feet away (Figure 4.6). Indeed, it was further noted, the transmissivity and storage coefficient resulting from the concurrent pump test would prove invaluable should the initial hypothesis not be supported by the chemical analyses. The storage coefficient, for example, might indicate whether the aquifer is under artesian conditions and thereby somewhat protected from downward migration of contaminants from surface activities by an impermeable confining stratum. Dense glacial till layers ('hardpan' in drillers' parlance) had been encountered in drilling each well, though the location and thickness of the till were substantially different for Well H-1 vs. Well H-2 (Figure 4.7); both wells had been constructed by the same driller and tested by the same engineer (Robinson, 1951 and 1959).

4.3.3 Field Investigation

The proposed combined chemical time-series study and pumping test was conducted by the author on July 24, 1981. Well H-1 and Well H-2 had been inoperative for 12 hours and 72 hours, respectively, prior to commencing the tests to allow the aquifer to locally resume its original static water-level elevation(s). To initiate the tests, the static water levels in both wells were measured and the pump in Well H-1 was started. Water-level elevations in both wells were subsequently measured at 1,5,10,30,60,90,180,and 300 minutes of pumpage, and the discharge from Well H-1 was maintained at a constant flowrate of 1,175 gallons per



(after Robinson, 1951 & 1959)

Figure 4.7. Comparison of Driller's Logs for Lakewood Water District Wells H-1 and H-2 (Data from Robinson 1951 & 1959)

minute. Samples for volatile organic analyses were taken in duplicate from a tap at the well head of Well H-1 concurrent with the water-level measurements.

Sample collection and handling protocols for volatile organic analyses of drinking water were still evolving at EPA at the time of the test, but the general trend was to adhere as closely as possible to procedures outlined by EPA's Sampling and Analysis Procedures for Screening of Industrial Effluents for Priority Pollutants, issued by the Agency as a result of the consent decree of NRDC vs. Train. That is, specially prepared 40-mL glass vials with Teflon-faced septa in screw closures were filled so that air bubbles were excluded. The samples were immediately tagged, listed on field data sheets and chain of custody forms, and stored on ice for same-day transport to the Region 10 EPA Laboratory in Manchester, Washington. There they were analyzed by GC-Hall Conductivity Director and GC-MS instruments.

Lakewood Water District had expressed an interest in determining the quality of water discharged from its alternate supply well, Well H-2, so the pump in that well was started a few minutes after the last (300-minute) measurement and samples were taken from Well H-1. Because it seemed useful from a geohydraulic standpoint to allow Well H-2 to attain equilibrium conditions and gain insights from its hydraulic interference with Well H-1, measuements of water levels in both wells were taken at 1,5,10,and 30 minutes since the onset of pumpage at Well H-2, with its discharge at a constant 875 gallons per minute. By that time, equilibrium conditions were apparent (Figure 4.8), so the tests were terminated by shutting down both wells.

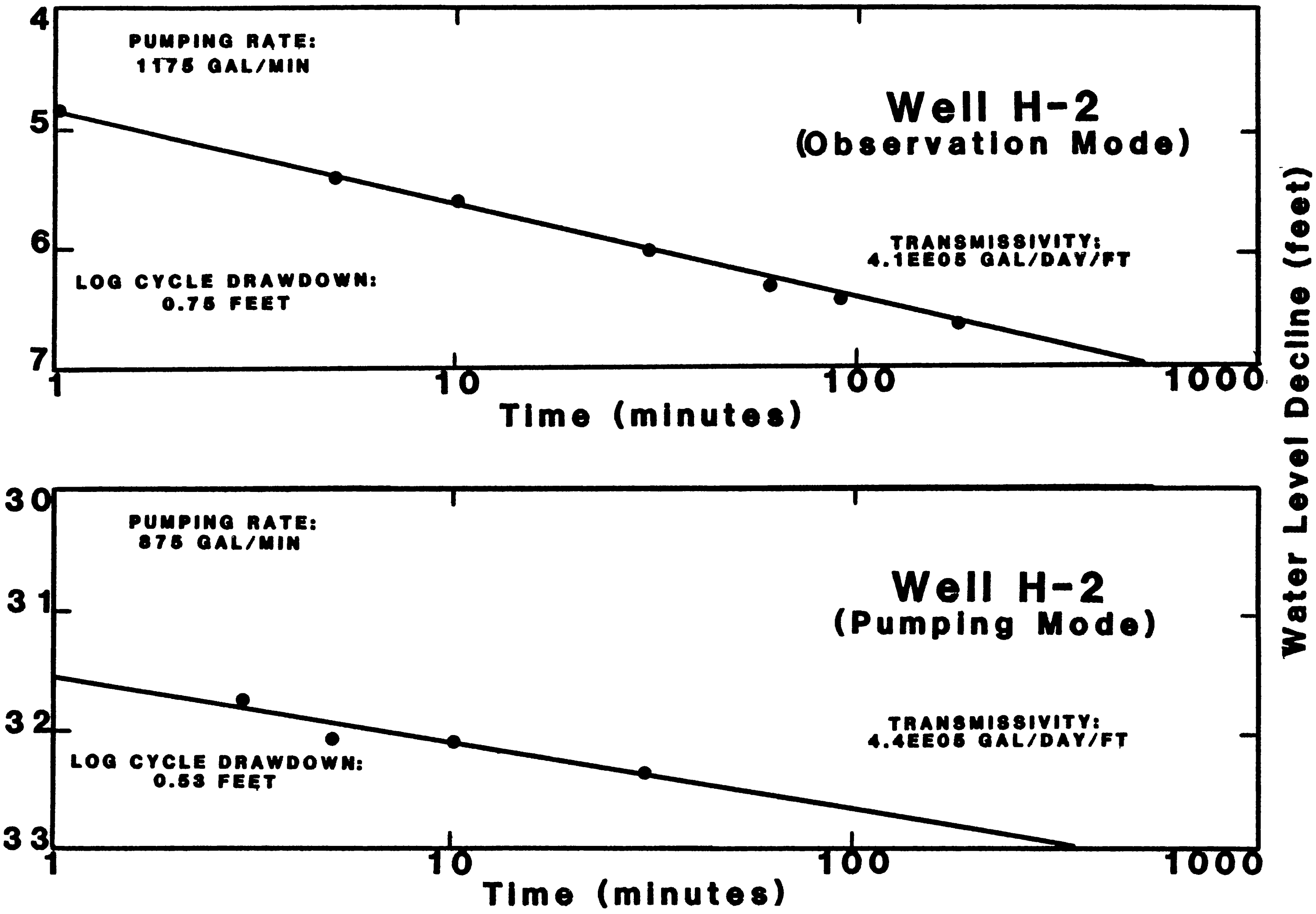


Figure 4.8. Semi-Log (Cooper-Jacob) Drawdown-Time Plots for Pump Tests of Lakewood Wells, July 24, 1981

Samples for volatile organic analysis were collected from Well H-2 concurrent with the water-level measurements. The reasoning behind this was that it would be useful to provide a chemical time-series pattern from Well H-2 for comparison with the data obtained from Well H-1. The nature of the contaminant source might be more readily discernible by such a comparison, should the contaminants not originate inside Well H-1. Well H-2 had not undergone maintenance similar to Well H-1, so there was no reason to believe it might have residues of solvents as a result of cleaning its pump column, etc.

4.3.4 Experimental Results

Figure 4.8 shows that the transmissivity of the aquifer is approximately 4.1×10^5 gal./day/ft. (5.5×10^4 sq.ft./day) using the Cooper-Jacob's equilibrium form of the Theis equation to evaluate drawdowns observed in Well H-2 while Well H-1 was pumping. The virtual lack of roll-on or significant slope changes is justification enough for assuming that equilibrium conditions prevailed; the high-transmissivity value and low storage coefficent (.0001) also lend support. The well function argument, **u**, (= $r^2S/4Tt$) is equal to .000014, so the Cooper-Jacob approximation of the Theis equation is valid for these data (recall that **u** should be less than 0.01). Moreover, a best-fit plot of additional drawdown generated by Well H-2 itself during subsequent testing (also Figure 4.8) yields a transmissivity of 4.4×10^5 gal./day/ft. (5.9×10^4 sq.ft./day), in good agreement with that from Well H-1 pumpage.

During construction of Well H-1, the transmissivity estimate had been shown to increase rapidly due to development of surging and overpumping (Robinson 1951). At the time it was put into service, the transmissivity was estimated to be 4.1×10^5 gal./day/ft. (5.5×10^3 sq.ft/day). In the course of constructing Well H-2 some eight years later, it was noted that the capacity of Well H-1 substantially increased during efforts to develop Well H-2 by overpumping; Well H-1 was noted to pull dirty water for a brief period at that time, presumably by removing fines loosened from the work on Well H-2 (Robinson 1959). The USGS (Griffin and others 1962) analyzed pumping test data for Well H-1 with its increased capacity and estimated transmissivity to be 9.3×10^4 gal./day/ft. (1.2×10^4 sq.ft./day). It would seem then that continual removal of the fines from the glacial washout aquifer over the years has resulted in raising the estimated transmissivity value based on Well H-1 pumpage from 5.5×10^3 sq.ft./day to 5.5×10^4 sq.ft./day, or one order of magnitude. Regardless of which estimate is held to be most representative of local conditions, the aquifer is certainly highly transmissive, as expected from the lithology of the production zone described by the drillers (Figure 4.7).

The USGS (Griffin and others 1962) presented a generalized water-table contour map for the Tacoma-Lakewood area which indicated that the local gradient at the Lakewood wells was approximately 25 feet per mile. The data generated in this study suggest it may be as low as 10 feet per mile locally. Using these bounds, an average transmissivity of 5×10^4 sq.ft./day, and estimated effective porosity of 0.25 and a saturated aquifer thickness of 100 feet, the ground-water flow velocity locally would be somewhere between 4 and 10 feet per day. The observations made during this study concerning the direction of flow agree with those of Griffin and others (1962); it is towards the northwest (magnetic bearing approximately 295 degrees). Well H-2 lies north-northwest (magnetic bearing 320 degrees) of Well H-1, so that ground-water generally proceeds from Well H-1 towards Well H-2 in the absence of pumping. As will be shown later, this is a significant factor in interpretation of the chemical time-series results for source identification, because of the upgradient-downgradient relationship between the two wells.

Figure 4.9 is a graphical presentation of the concentrations of the volatile organics found in the samples collected from Well H-1 on July 24, 1981. It is quite clear that the contaminant concentrations did not rapidly decrease, as postulated for internal contamination of Well H-1 by solvent residues. On the contrary, the contaminant concentrations rapidly increased and appeared to level off after only a few hours of pumpage. These chemical time-series patterns would seem to implicate an external source of contamination.

Figure 4.10 gives similar indications for Well H-2, but at markedly higher initial and subsequent concentrations, Does this mean that the source is closer to Well H-2? A hydraulic divide was formed by operating both wells simultaneously while sampling Well H-2, but no samples were taken from Well H-1 during that portion of the test; 20-20 hindsight suggests that such samples would have been invaluable. Given the data limitations, conclusions could not, at that time, be drawn as to whether the contaminants lay primarily on one side of the hydraulic divide or the

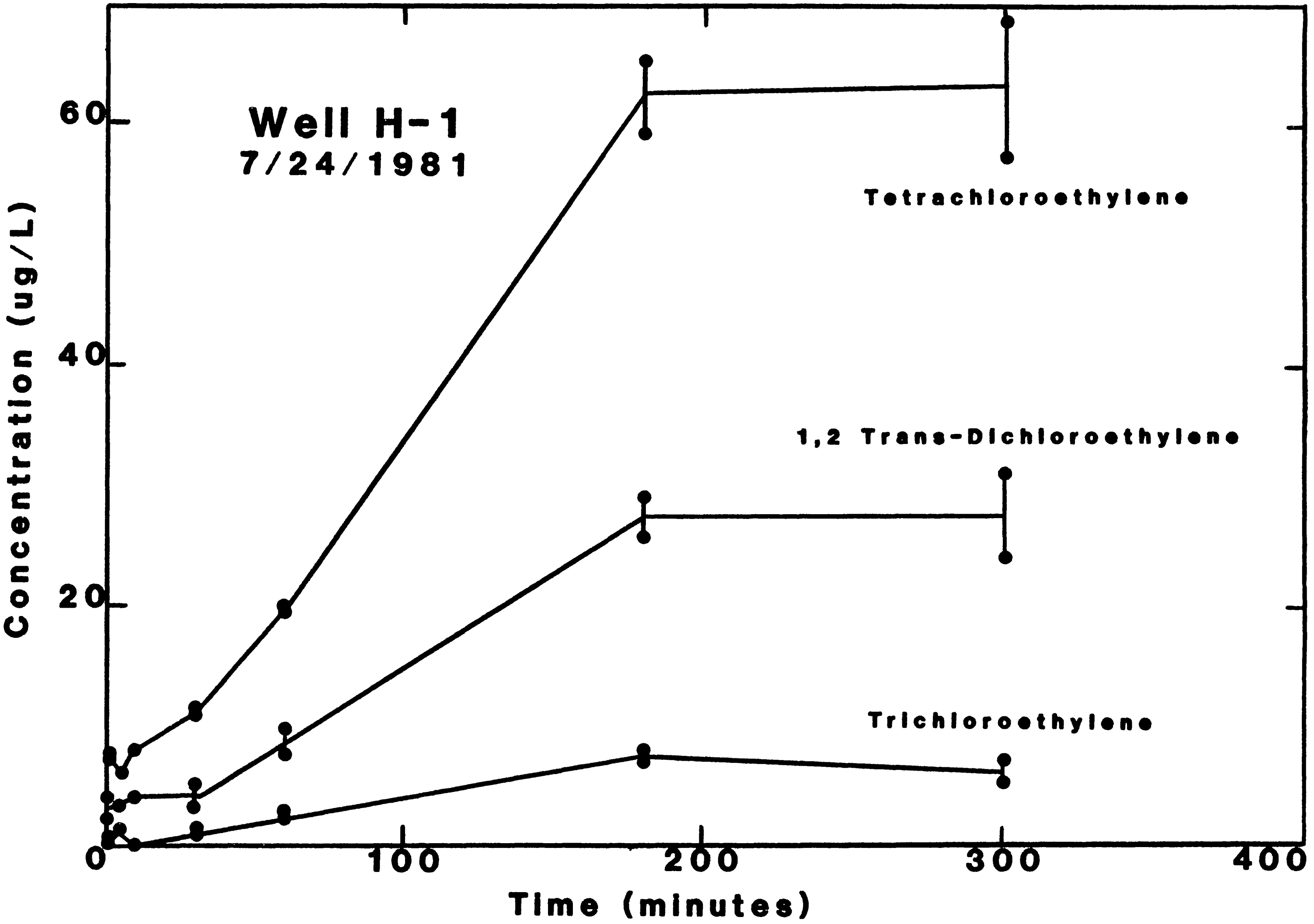


Figure 4.9. Chemical Time-Series Plots for Samples Collected from Well H-1 During July 24, 1981 Pump Tests

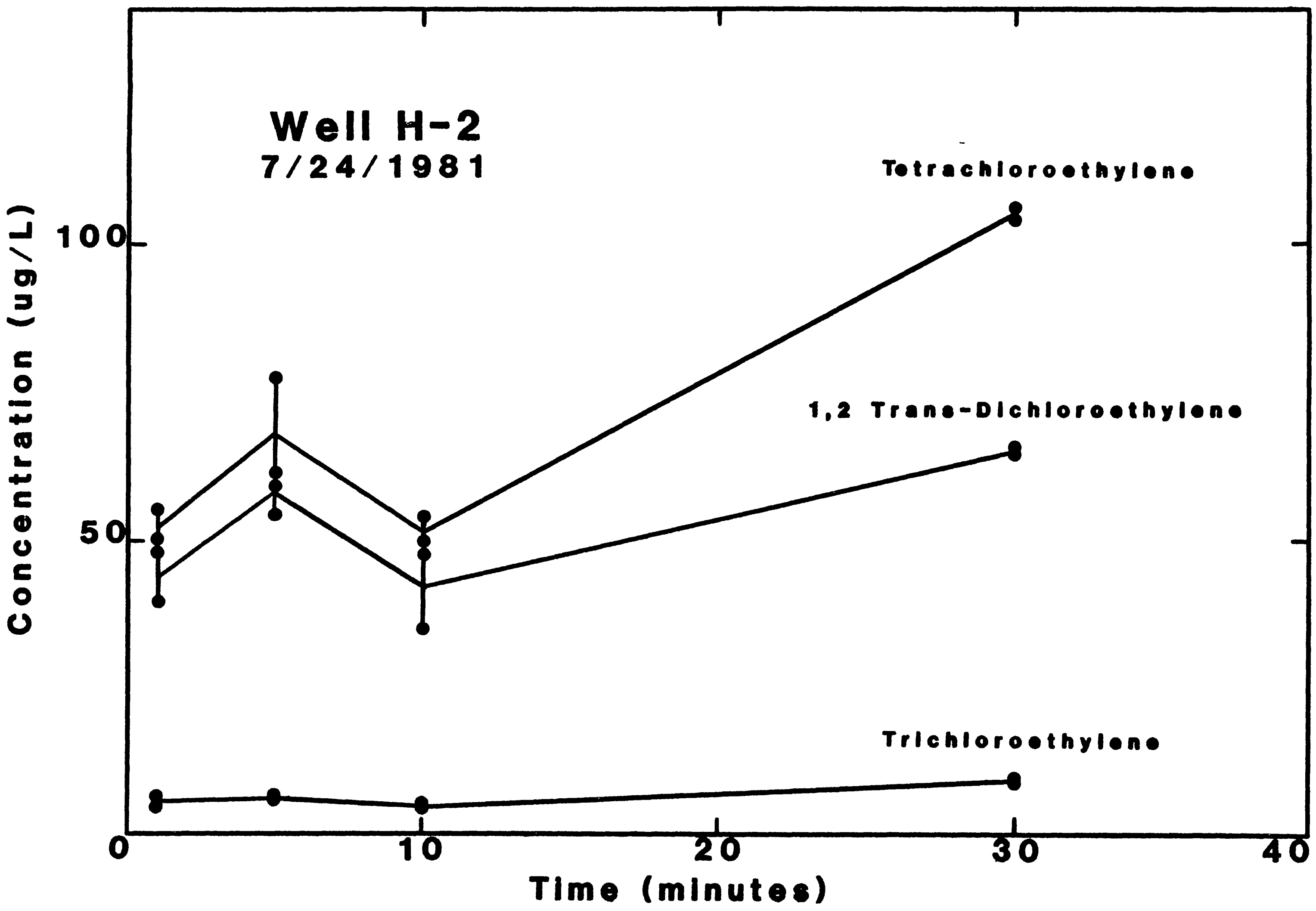


Figure 4.10. Chemical Time-Series Plots for Samples Collected from Well H-2 During July 24, 1981 Pump Tests

other--a piece of information that Lakewood Water District could use to help manage its water quality. If contaminants lay primarily on one side of the hydraulic divide, a temporary solution would be to use water discharged from the uncontaminated well for supply purposes and waste water discharged from the more contaminated well.

To resolve this uncertainty, a follow-up sampling was conducted August 8, 1981. Both wells had been inoperative since the previous test and were started simutaneously. After 10 hours of pumping, duplicate volatile organic analysis samples were obtained from each well (Table 4.1). The picture now seemed quite clear: the higher, most troublesome levels of the contaminants were available only to Well H-2. Further reflection on the ramifications of these data and the July 24, 1981, study suggested that the source of the contaminants was not only on the Well H-2 side of the hydraulic divide, but was probably downgradient of Well H-2 itself.

Such a source location would allow for the obvious differences in contaminant concentrations, and would also explain the high initial levels with the onset of pumpage at Well H-2 after pumping Well H-1 for several hours. The contaminants would be drawn from beyond Well H-2 towards Well H-1, bathing Well H-2 in higher concentrations with increased contaminant levels. It would not be so coincidental, then, that the initial contaminant levels in Well H-2 should be so near to the final concentrations in Well H-1 (compare Figure 4.9 with Figure 4.10). A hypothetical illustration of this mechanism of contaminant distribution has been given earlier (Keely, 1982); it has been modified here to lend more specificity to the example being discussed (Figure 4.11).

Table 4.1. Select Volatile Organic Chemical Concentrations (µg/L) in Wells H-1 and H-2 on August 8, 1981.

	Well No.	
Contaminant	**Well H-1**	**Well H-2**
Tetrachloroethene	31	196
Tetrachloroethene (dup)	43	272
Trichloroethene	3.8	20
Trichloroethene (dup)	3.6	15
trans-dichloroethene	6.5	101
trans-dichloroethene (dup)	15	86

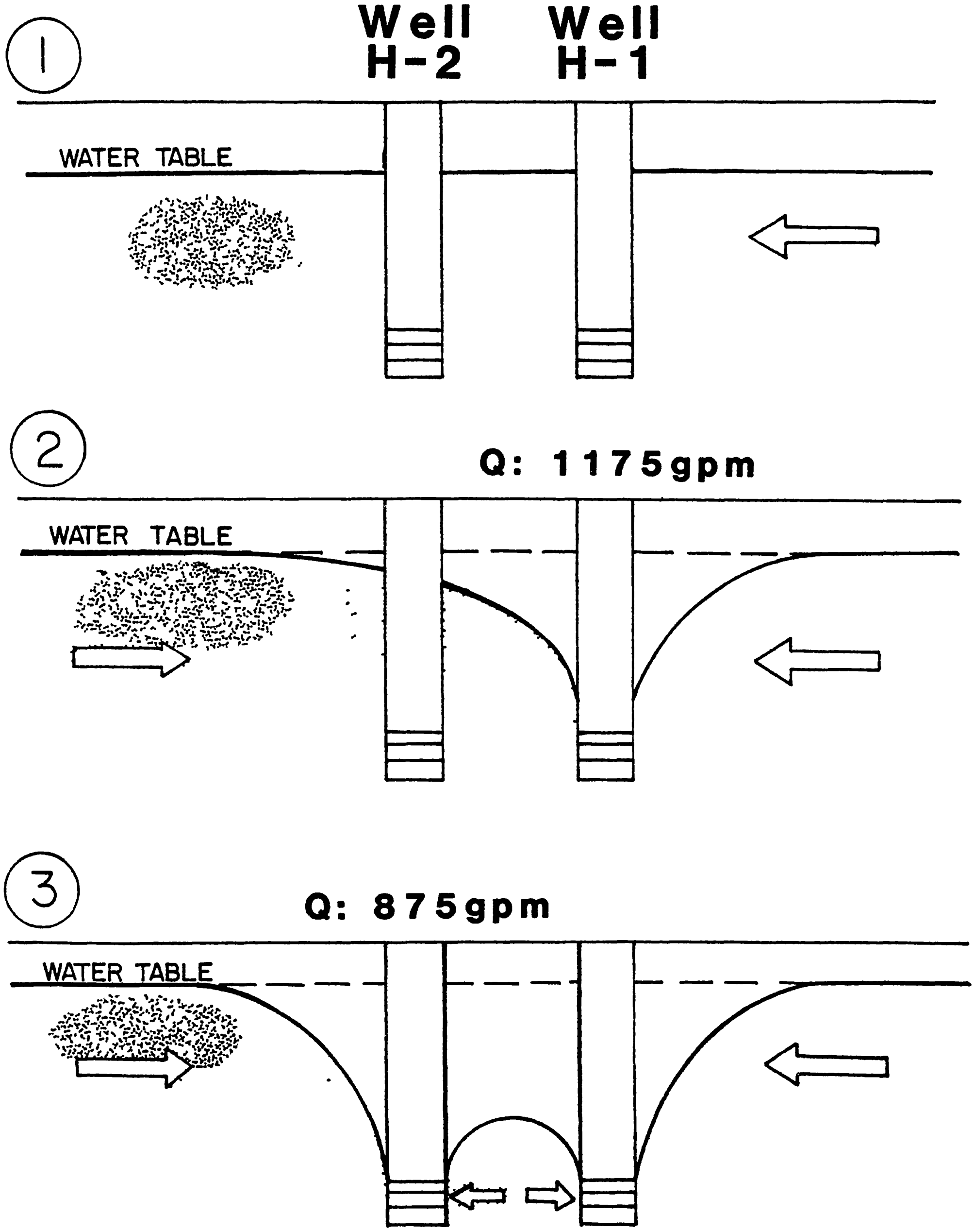


Figure 4.11. Mechanism of Contaminant Plume Isolation by Hydraulic Interference Between Wells H-1 and H-2

Since the aforementioned scenario was not the only one which could reasonably give results observed during the tests, other hypotheses were advanced. In particular, it was noted that a perched water table would probably lay atop the semi-confining till layer (assumed to be unsaturated) and that Well H-2 did not have the same annular grouting through the till layer that Well H-1 did. Instead, Well H-2 had a pea gravel jacket surrounding the casing, which was installed because of excessive caving of the formation while pulling back temporary drive casing (Robinson 1959). The EPA Regional Geologist, Jack Sceva, had mapped the geology of the local area several years before, while working for the USGS. He noted that it was not uncommon to find significant quantities of water in perched zones, and given the pea-gravel jacket that extended from the well screen to within a few feet of the ground surface at Well H-2, he postulated that the perched zone might be a highly localized source providing the contamination. In order to test either hypothesis, monitoring wells had to be installed.

Figure 4.6 shows the relative positions of Well H-1, Well H-2, several shallow wells installed to test the quality of the uppermost portion of the saturated zone and several deep wells installed to test the quality of the production zone. During the course of field investigations at the Ponder's Corner site, a number of cost-saving sampling alternatives were chosen. These related principally to the field use of a portable gas chromatograph (Organic Vapor Analyzer) for the screening of water samples and soil extracts taken while drilling monitoring wells, and to the use of selective analyses (volatiles only) of ground-water samples when initial results showed only a narrow group of contaminants to be present. The lowered analytical costs, in part, allowed for increased expenditures for geotechnical characterization of the site. The monitoring wells were installed under the supervision of the Ecology and Environment field team, who also conducted detailed pump tests and additional chemical time-series samplings (Wolf and Boateng 1983). These pump tests confirmed earlier indications of the high transmissivity and semi-artesian nature of the production aquifer.

The pump tests that were conducted at this site not only yielded estimates of local transmissivity and storage coefficients, but also confirmed the presence of a major aquifer boundary nearby; a buried glacial till drumlin just west of the site parallels the general direction (north) of regional flow. The pump tests clearly showed some anisotropy of the sediments as well; drawdown contours produced an elliptical cone of drawdown, the major axis of which was aligned with the regional flow to the north. This information resulted in modifications to the original plans, which called for drilling and constructing several monitoring wells west of the site. Instead, more monitoring wells were drilled along the north-south axis. The hydrogeologic parameter estimates obtained from the pump tests strengthened the conceptualization of contaminants being drawn back against the regional flow because the capture zones of the pumping wells were sufficiently distorted by the local anisotropy to more than encompass the contaminant source.

4.3.5 Monitoring Well Drilling Procedures

It is useful to explain how difficulties with the hollow-stem augering method led to the adoption of the cable-tool drilling method for the installation of all monitoring wells in subsequent studies of this site. Problems in obtaining sediment-free water samples when constructing monitoring wells by the hollow-stem augering method were encountered in the preliminary study. It is believed that this was the result of limitations involved in the emplacement an adequate gravel pack through the hollow-stem auger. A four-inch diameter hollow-stem auger was used to drill to the target depth, and a two-inch diameter monitoring well was then placed inside. The one-inch wide annular space that remained was used to conduct gravel to the target zone. Because too large a gravel (3/8 inch pea gravel) was used on the first few attempts, it bridged in the annular space and caused the entire monitoring well to be withdrawn along with the auger in the final stage of well construction. When finer gravels and sand were used as the packing material, they tended to slough away from the wellbore; as the auger was withdrawn the borehole immediately collapsed around the well so that an incomplete gravel pack resulted.

Attempts to develop these wells were confined to air-lifting by compressed air injected through a small diameter pipe. It was possible to nearly evacuate a new monitoring well in only a few minutes by this technique, which was repeated several times to ensure delivery of sediment-free samples. The wells installed in that manner, however, accumulated several inches of sediment between sampling events, as determined by lowering depth sounding weights and bailers. Excessive purging of those wells was therefore necessary prior to each sampling. Consequently, it was determined that the cable-tool drilling method should be used. Installation of monitoring wells by this method was easily accomplished because of the use of a large diameter (eight-inch) temporary casing, which allowed the emplacement of a thick, stable gravel pack. The resulting wells were readily developed and yielded very little sediment between sampling events.

Prior to sampling, measurements of temperature, pH, and specific conductivity were obtained to help determine the adequacy of purging. These gross parameters stabilized very quickly; usually requiring only a few borehole volumes. Contaminant concentrations were also measured, to see if they would behave similarly. The results, however, showed that contaminant concentrations do not necessarily stabilize as do gross physical and chemical parameters (Figures 4.9 and 4.10; also see Figures 4.12 and 4.13 later in this section). This means that attempts to use stabilization of gross parameters as a yardstick by which to ascertain the adequacy of purging efforts must be tempered with the realization that tough choices still remain regarding the meaning of values reported for the contaminants, as discussed in the preceeding chapter.

4.3.6 General Indications

Two years and some 33 monitoring wells after the investigation was first initiated, the investigators have concluded that the glacial till layer is discontinuous, but is saturated and has localized ground-water lenses overlying it and these lenses are intimately hydraulically connected with the production zone. The contaminants affecting Well H-1 and Well H-2 do not originate in the shallow lenses atop the glacial till in the immediate vicinity of these supply wells.

Rather, the contaminant source is approximately 475 feet away along a true bearing of 12 degrees (351 degrees magnetic, with 21 degrees easterly variation locally) from Well H-2, or 400 feet downgradient and 260 feet cross-gradient. There, septic tank effluent rich in solvents from a dry cleaning establishment has been filtering down through the discontinuous semi-permeable glacial till to reach the production aquifer. The pumping action of the Lakewood supply wells has been sufficient to locally reverse the natural gradient, causing the contaminants to move upgradient and cross-gradient with respect to pre-pumping conditions. Table 4.2 is a summary of the most recent sampling of all wells for volatile organic analyses. By comparison of the data presented there with the generalized well locations shown in Figure 4.6, it is evident that the only monitoring wells with detectable contaminant levels are those which are immediately adjacent to and downgradient or cross-gradient from the pumping wells (monitoring wells no.'s 11,24,15,16,20,21).

4.3.7 Distances of Draw and Capture Zones

The methods presented by Keely (1982) and Keely and Tsang (1983) for estimating contaminant travel in response to pumpage were used to check initial time-series test interpretations and the assertion that either supply well could locally reverse the natural gradient and cause

Table 4.2. Select Volatile Organic Chemical Concentrations (µg/L) in Lakewood Study Wells from 1983 Survey.

Date	Well No.	DCE	TCE	PCE
02-02-83	1	1.5	1K	1.6
02-03-83	2	---	---	---
02-01-83	3	---	---	---
01-31-83	4	1K	---	1K
01-31-83	5	---	---	---
02-03-83	6	1K	---	1K
02-02-83	7	---	---	---
02-01-83	8	1K	---	1K
02-01-83	9	---	---	---
01-31-83	10	---	---	1K
02-02-83	11A	9.2	---	1K
02-02-83	11B	5.6	1K	1K
02-03-83	12	---	---	---
02-01-83	13A	---	---	---
02-02-83	13B	---	1K	---
02-03-83	14	---	---	---
02-03-83	15A	26	4.0	96
02-03-83	15B	100	4.7	60
02-04-83	16A	188	6.7	76
02-04-83	16B	14	1K	4.0
02-07-83	17A	---	---	1K
02-07-83	17B	---	---	---
02-23-83	18	---	---	1K
02-23-83	19A	---	1K	1K
02-23-83	19B	---	---	1K
02-04-83	20A	179	23	632
02-04-83	20B	485	43	362
02-07-83	21	41	1.9	19
02-03-83	22	---	---	---
02-03-83	24A	202	22	522
02-02-83	24B	224	25	517
02-23-83	25	---	---	---
02-02-83	H-1	268	26	417
02-23-83	H-2	220	25	345

Notes: DCE is trans-dichloroethene.
TCE is trichloroethene.
PCE is tetrachloroethene.
1K indicates detection above the sensitivity level, but not quantifiable.
--- indicates below detection level.
Well 23 does not exist; it was abandoned at the property owner's request.

effluent-contaminated water to flow 400 feet upgradient and 260 feet cross-gradient to the pumping well(s). The first of two methods, distance of draw estimates, ignores the effect of the natural ground-water flow velocity on the manner in which water is drawn into a well by pumping; it assumes equal velocity of travel radially towards the well at any select distance (as discussed in the preceeding chapter). This is an acceptable approach for crude estimation, particularly when reliable data are not available as to the direction and magnitude of the natural flow velocity. The author notes that the method is a rough estimation technique only (Keely 1982), and most useful for short pumping periods where the estimated distance of draw (contaminant travel) falls considerably short of reaching the point where the drawdowns caused by pumpage are offset by the natural gradient (e.g., velocity of pumping = natural velocity).

For the Lakewood wells this method was employed to give a first order approximation of the average distance traveled by the contaminants during the 300-minute chemical time-series test of Well H-1 on July 24, 1981. Assuming an effective porosity of 0.25 and an effective aquifer thickness of 65 feet (tight till and silts occupy 35 feet of the 100-foot saturated thickness), this calculation resulted in an estimated value of 30 feet traveled during the test. Given the maximum natural flow velocity of 10 feet per day described earlier, Well H-1 could have been inoperative at least three days prior to the test and still pulled in some contaminants (10 feet/day times 3 days = 30 feet). The best estimate of natural flow velocity (at 4 feet per day) and the actual down time of 12 hours implies that contaminants may have been as close as 2 feet downgradient of Well

H-1 (4 feet/day times 0.5 days = 2 feet). It would then have pulled in contaminants almost immediately, in agreement with actual observations.

The second method of estimation incorporates the effect of the natural flow velocity on pumpage patterns and contaminant travel times; it yields a distribution plot of the velocity field for the combination of pumping and natural flow velocities. Using this method, estimates of the downgradient stagnation point and cross-gradient lateral limit of the capture zone (portion of the aquifer contributing water to the well) of each pumping well were prepared. For Well H-1, the downgradient stagnation point was estimated to be 554 feet, assuming natural flow velocity of 4 feet per day. The lateral limit to either side of the well is a maximum of π times the stagnation point value, which equals 1,740 feet. The corresponding values for Well H-2 were 412 feet to the drawgradient stagnation point, and 1,295 feet to the maximum lateral limits of the capture zone. The contaminant source, which is only 400 feet downgradient and 260 feet cross-gradient from Well H-2 could easily be drawn to either well. To underscore the implications of these conservative/idealized estimates, recall that the generally anisotropic nature of these glacial deposits, (as previously described by Walters and Kimmel, 1968) was particularly pronounced locally, with the major transmissivity axis generally oriented parallel to the natural flow direction (Wolf and Boateng,1983).

4.3.8 Contaminant Peculiarities

The source of contamination, a septic tank at a dry-cleaning facility, was found to have received large amounts of tetrachloroethylene and trichloroethylene, but no known amounts of cis- or trans-dichloroethylene; whereas the contaminated wells had relatively high concentrations of both dichloroethylene isomers. Initially it was thought that other sources might also be present and would explain the high concentrations of these contaminants. However, it soon became clear, by the distribution of contaminants and the relative locations of other potential sources, that research results regarding the potential biotransformations of tetrachloroethylene and trichloroethylene (Wilson and McNabb, 1981) would more satisfactorily explain the observations.

Simulations of this kind of problem could be adequately performed only by contaminant transport models capable of incorporating the effects of the pumping wells on the regional flow field. More sophisticated approximations would also require the ability to account for the anisotropic and heterogeneous character of the site, the retardation of volatile organic contaminants by sorption, and their possible biotransformations. Given the highly localized nature of the contaminant source and limited extent of the plume, however, there was insufficient justification for pursuing such efforts. The resolution of the problem was possible by relatively simple source removal techniques (excavation of the septic tank and elimination of discharges).

4.3.9 Tracers and Time-Series

The use of chemical time-series sampling is quite similar to the use of tracer tests to gain insights into preferential paths of contaminant movement. Davis and others (p.14, 1980) define a tracer as:

> "matter or energy carried by water which will give information concerning the direction and/or velocity of the water, as well as potential contaminants which could be transported by the water...A tracer can be entirely natural such as the heat carried by a plume of geothermal water, it can be accidentally introduced such as chlorides leached from landfill, or it can be introduced intentionally such as dyes placed in sink holes..."

Since the time of release and the input concentration of intentional tracers are known for traditional tracer tests but unknown for contaminant studies, the interpretation of data from chemical time-series sampling might initially seem to be severely hampered. However, it is the pattern of relative concentrations over time which are actually examined in either case. The absolute values are neither required nor are they all that readily obtainable even under the best controlled field tests. There are simply too many avenues for tracer losses (sorption, ion-exchange, biodegradation, etc.) to allow for proper mass balancing.

Preferential paths of flow and contaminant transport are determined by frequent samplings at several locations regardless of whether one is conducting a traditional tracer-test, a conventional contaminant investigation or a chemical time-series sampling study. The prime difference between conventional surveys and the approach described here is simply timing. One can take the conventional approach of sampling an array of monitoring wells several times over a period of months, allowing the natural system to slowly move a contamiant plume, or one can accelerate the natural system by pumpage and gain similar information by rapidly collecting a succession of samples.

There are sound benefits to incorporation of successive pumped samples in ground-water contaminant investigations other than temporal sampling efficiency, however. It has been shown here that concurrent hydraulic testing not only yields information about aquifer parameters, but may be readily adapted to enhance the use of time-series analyses; e.g.,purposeful hydraulic interference can be used to isolate and locate a contaminant source.

4.3.10 Parting Shots

Given the difficulty of locating a contaminant source by nonpumping monitoring well installations alone, it now seems that greater confidence in the chemical time-series patterns and associated hydraulic testing was warranted for the Lakewood study. Still, hindsight should not lure one into a false sense of security and overconfidence. A prudent mixture of some nonpumping monitoring wells and pumping wells will always yield the best results. Consider, for example, the difficulty of interpreting Figures 4.12 and 4.13 without additional monitoring wells, which show the results of extended chemical time-series samples collected by the Ecology and Environment field team from Well H-1 and Well H-2 during August 1982 and February 1983, respectively. The concentrations rise as they did during the July 1981 samplings, but fall rapidly after a few hundred minutes of pumping.

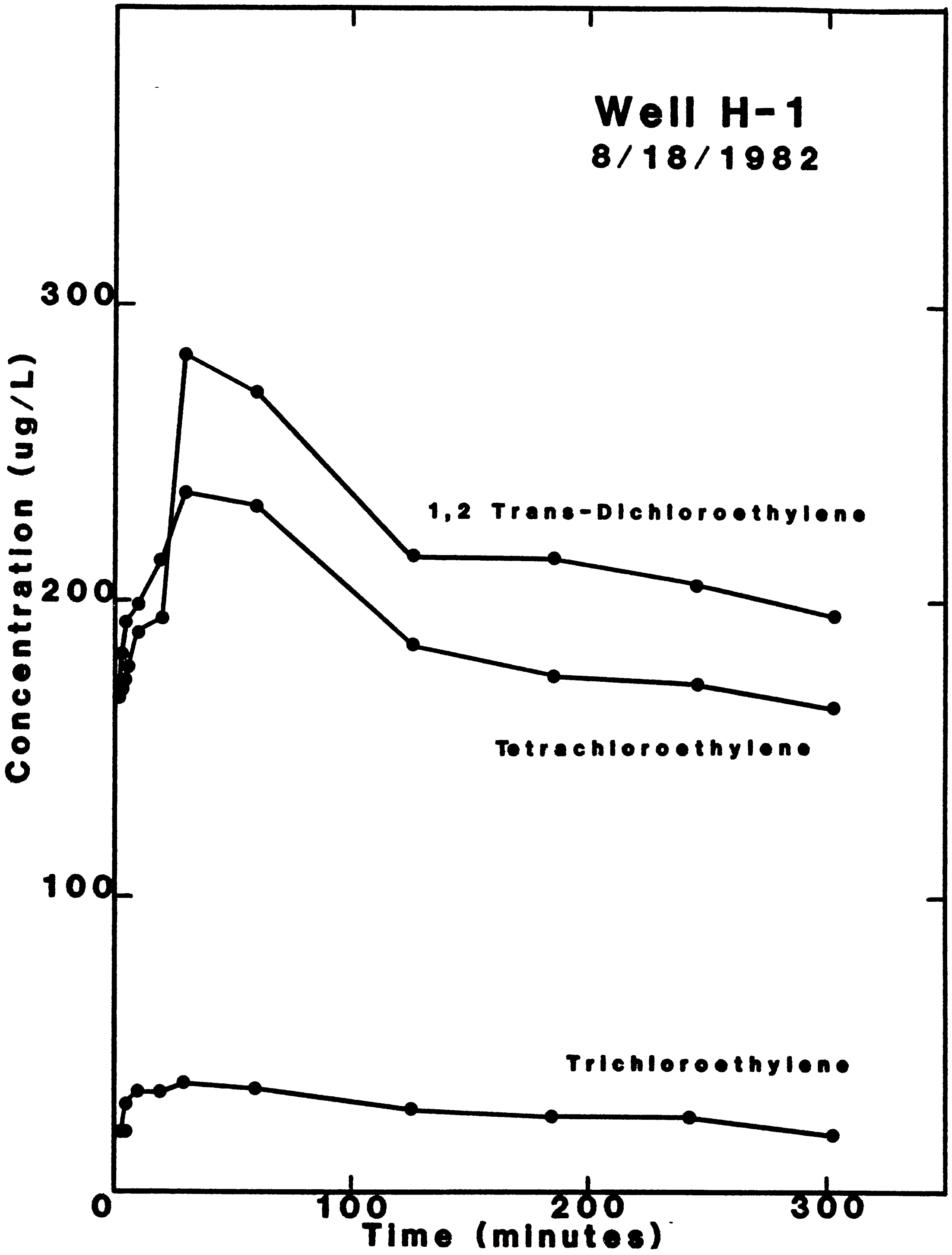


Figure 4.12. Chemical Time-Series Plots for Samples Collected from Well H-1 During August 18, 1982 Sampling

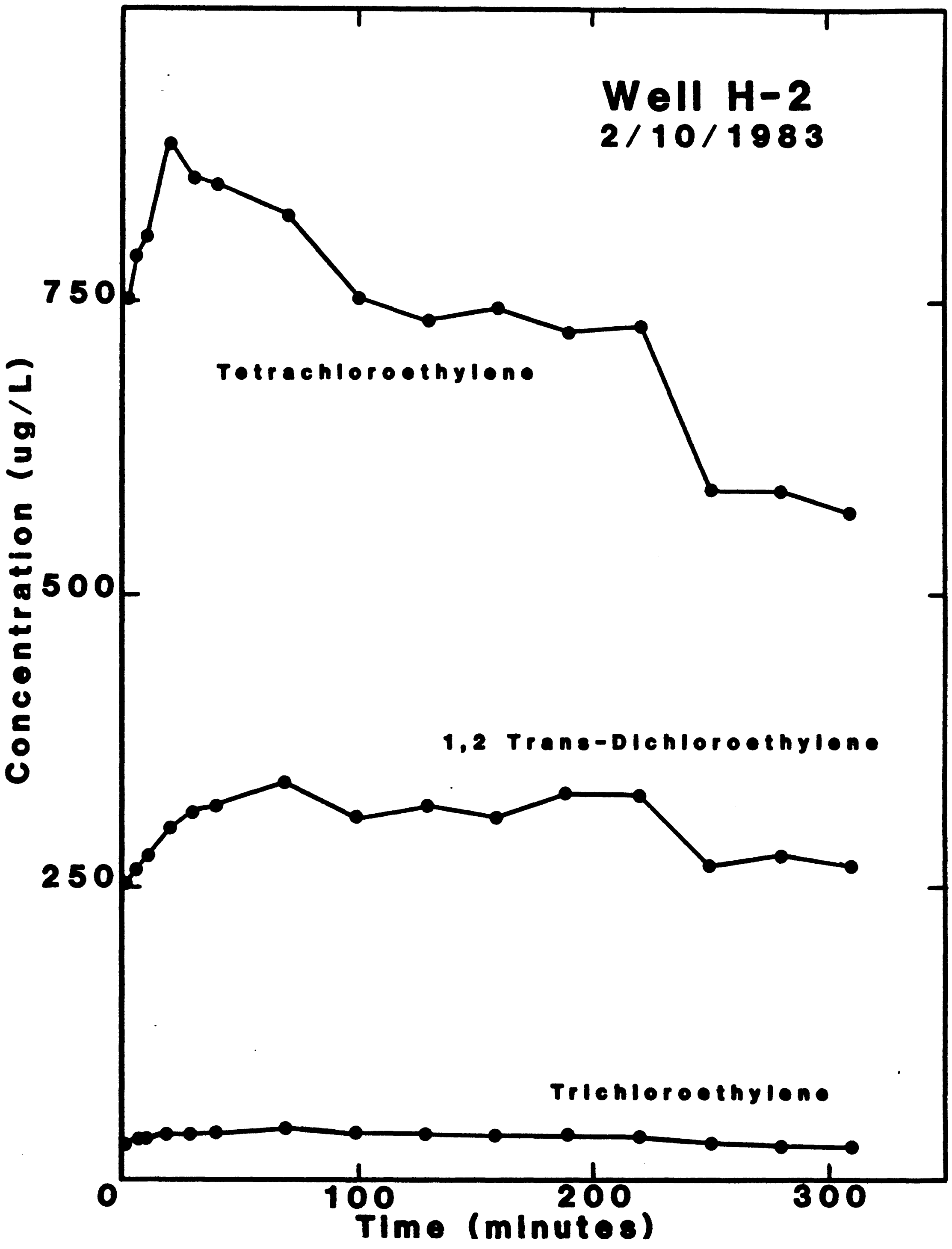


Figure 4.13. Chemical Time-Series Plots for Samples Collected from Well H-2 During February 10, 1983 Sampling

What are the implications of these curves? Is the transient nature of the contaminant loadings a result of sporadic release to the aquifer by occasional deep percolation of the dry cleaning septic waste, due to rainfall or irrigation events, similar to the cyclic effects described by Pettyjohn (1982)? Is it a matter of spatially varied sorptive properties of the aquifer due to changes in the particle sizes of the aquifer solids as suggested by Rea and Upchurch(1980), but which may also change with time because of the need to establish new equilibria with sorption sites accustomed to other species as suggested by Reardon (1981)? Perhaps biodegradation has taken its toll. Surely resolution of which mechanisms are governing the shape of the chemical time-series patterns in Figures 4.12 and 4.13 requires additional testing, that may include the installation of additional monitoring wells (pumping or not); those data are insufficient for stand-alone interpretations.

Such uncertainties and interpretive difficulties reduce chemical time-series sampling to what it truly is; simply another useful tool for the contaminant hydrologist, another method that may generate insights to help piece the puzzle together. It is not a panacea any more than the exclusive use of nonpumping monitoring wells is. It may result in increased efficiency and lowered economic costs for ground-water contamination investigation (Keely 1982), but it takes common sense and hydrologic intuition to be properly applied, just like any other investigative technique available today.

Case History No.3

4.4.1 History and Current Situation

A chlorine production plant that began operations in the mid-1960's is now engaged in trying to remediate a dense brine plume that contains significant concentrations of mercury. During the first decade of operation, rock salt was used as the feedstock for the 250 ton/day electrolytic chlorine production process at the plant. This generated a sand-like brine sludge that was partially dewatered and landfilled with other solid process wastes (e.g., spent graphite anodes) in a natural ravine onsite (labelled 'E1' in Figure 4.14) for about seven years. In response to changing state laws, solid wastes were disposed of in a one-acre clay-lined landfill onsite (labelled 'E2' in Figure 4.14) until the late 1970's. During that time, the disposal procedure for the brine sludge was changed to an automated system that piped the sludge in a slurried form to the northern half of a two-acre lined disposal pond that had been constructed onsite.

Because of the nearing closure of the solid waste landfill, the pond was divided into two areas; one to accept solid wastes (labelled 'E3' in Figure 4.14) and one to accept the slurried brine sludge (labelled 'E4' in Figure 4.14). Process brine was used as the slurrying fluid, which was decanted from the surface of the pond as the sludge settled, and pumped back into the plant process operation. Mercury is needed in the electrolytic chlorine production process and is a contaminant of the brine fluid and sludge as a result. Hence, all of the disposal areas

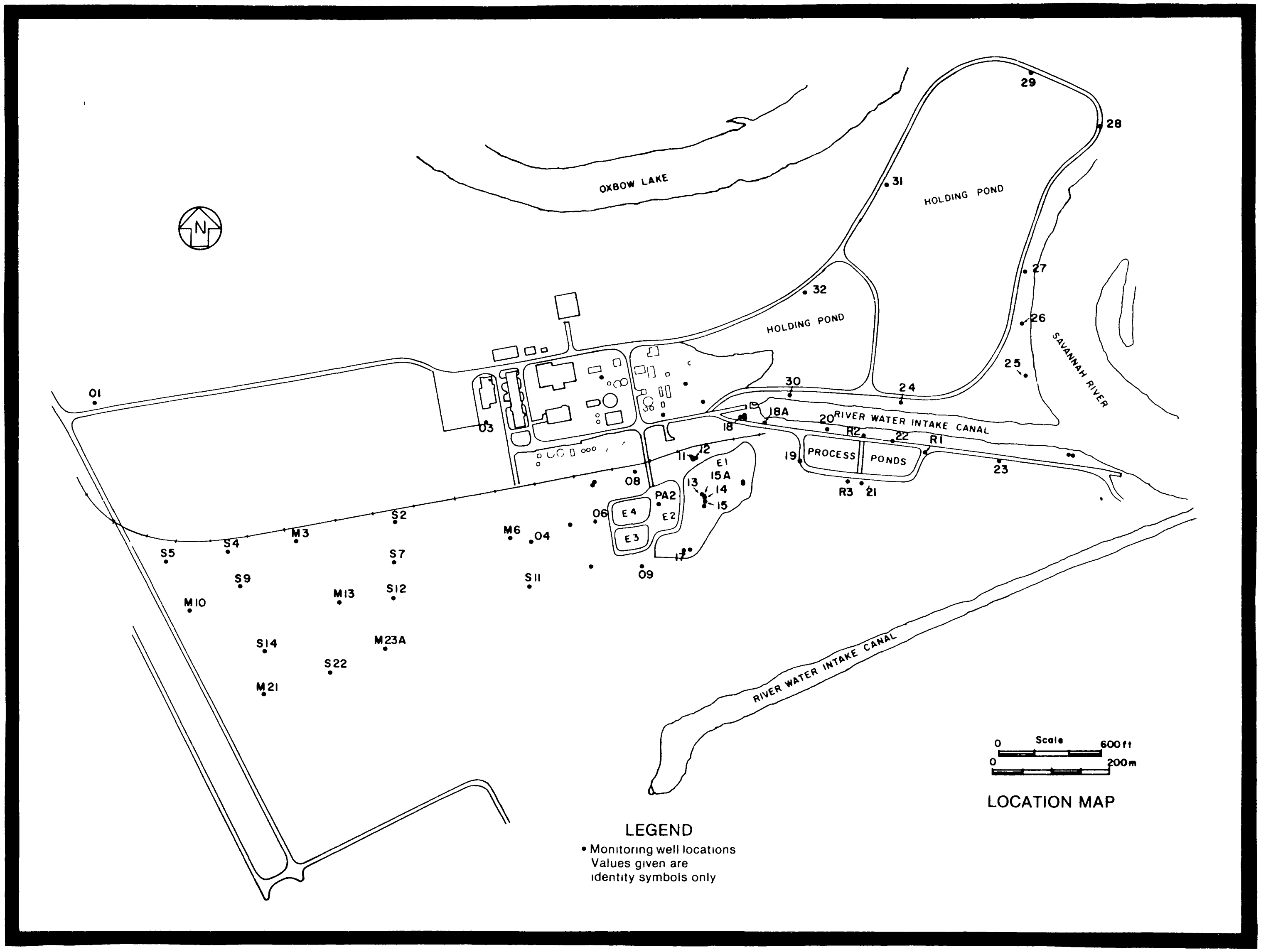


Figure 4.14. Monitoring Well Location Map for Case History No.3

posed the potential for contamination of local ground waters with substances regulated under state and national environmental laws.

Results of ground-water quality studies that began in 1980 have shown that the disposal pond is leaking and may have been leaking for a long time; relatively minor amounts of contaminants may also be leaching from the abandoned solid waste disposal areas onsite. The initial efforts to ascertain the extent of any contamination did not develop a sense of the true magnitude of this problem because only shallow wells were used and no consideration was given to the severe downward flow components that would be imparted by the highly dense nature of fluids in the disposal pond (e.g., 100,000+ ppm total dissolved solids).

A more intensive study was initiated in mid-1982, which included the installation of deeper monitoring wells and somewhat better definition of the lithology and stratigraphy of the alluvial sediments underlying the site. These efforts came about largely as a result of a 1982 Consent Agreement regarding findings of mercury at unacceptable concentrations in some of the shallow wells, and in preparation for establishing RCRA Part B permit conditions for a replacement disposal facility onsite (such permits require Corrective Action Plans for existing onsite contamination). The author became involved in the problem during the summer of 1982, as a technical reviewer of the initial study report (Olin Corporation, 1981) and potential expert witness for the state agencies handling the situation.

As a result of the review comments (e.g., the development of monitoring wells and the performance of an onsite 'pump test' were found to be wholly inadequate -- both having been done with a peristaltic pump capable of yielding far less than one gallon per minute!) and findings of mercury contamination in additional wells, a third major field effort was undertaken in the fall of 1982 (Olin Corporation, 1982). This included the installation of additional deep monitoring wells and many more split-spoon samples of subsurface solids for definition of the lithology and stratigraphy. By early 1984 (Olin Corporation, 1984), these efforts had shown that contamination extended from the surficial aquifer (comprised of two members, referred to as the 'alluvium' and 'Kt1' aquifers in the 1982 and 1984 reports) down through a clay aquitard ('Kt2') that the site operators had earlier tried to prove to be an impermeable barrier, through an underlying aquifer ('Kt3') they maintained could be minimally affected at the worst, through a second clay aquitard ('Kt4') that they insisted was absolutely impermeable in all of their reports, and into the major production aquifer below ('Kt5'); for a total of more than 200 ft. of downward travel. The plume has spread more than 1800 ft. horizontally in the surficial aquifer, but the lack of a sufficient number of deep wells has made conclusions about its lateral extent in the deeper strata premature.

At this time, efforts to design a remedy for the contamination are underway. The situation is still highly sensitive as the full extent of contamination in the deeper strata is not yet known and is a matter of extreme concern. The following discussion, then, relates to the author's review of reports that were available as of late 1985, and must be regarded in that light.

4.4.2. Quality of Background Data

The monitoring well construction, maintenance, and recordkeeping practices submitted by the plant were of poor quality and cast doubt on the ability of the monitoring well network to yield samples which accurately represent the occurrence and extent of contamination of the upper- and lower-aquifers. Specifically, rotary drilling with bentonitic muds or natural polymeric material (e.g.,'Revert') may have facilitated the movement of contaminants from one stratum to another (by cross-contamination within the boreholes), and may have resulted in artificially low levels of mercury being detected (due to sorption of mercury by the drilling fluid residues, and due to differential clogging of some strata by unrecovered drilling mud). Reports indicate that some monitoring wells have not been maintained in adequate condition and that confusion exists over the identity of others; these accounts connote an inadequate quality assurance program.

The sampling and analysis techniques employed were substandard and may have resulted in artificially low levels of mercury being detected and reported. Lack of filtration and acidification of samples immediately upon collection in the field allows many metals to precipitate or coprecipitate out of solution. One consequence is that there will be much higher concentrations reported for total mercury than for dissolved mercury despite the absence of particulate matter in any water sample (due to absorption of elemental mercury into the sample container walls). EPA's official Methods for Chemical Analysis of Water and Wastes (1979; Table 1) specifically recommends that the proper sample preservation for dissolved mercury is "Filter on site [and acidify with] HNO_3 to pH < 2" in recognition of the rapidity of precipitation reactions of metals. The official recommendation for determination of total mercury is similar; only the filtration step is omitted.

Generally, both dissolved and total mercury should be determined on samples obtained in ground-water investigations, due to the probability of detecting significant reserves of mercury sorbed to microfine particulates/aquifer solids in the unfiltered samples (in chemical equilibrium with the dissolved mercury). Otherwise, an estimate of the reserves of mercury in the system may be quite difficult to make; mercury in the presence of high chloride contents can be present as complexed species (e.g., $HgCl^{2-}$) having considerably different sorptive behavior in natural materials than elemental (Hg) and ionic mercury (Hg^{2+}).

Mercury analyses performed by the cold-vapor method used by the plant's laboratory analysts are subject to interferences from solutions containing appreciable ammounts of salt, causinq a loss of sensitivity and precision despite incorporation of background correction methods. This results in an inability to distinguish between 'true' background concentrations of mercury and much higher levels. The Method of Standard Additions is recommended as a means of determining the degree of interference and more accurately estimating the true value.

The pH determinations made during the studies are of questionable accuracy; they were not performed immediately after collection of each sample (onsite) as is recommended by EPA (1979; Table 1). Other chemical analyses to date primarily consist of chloride determinations. These are inadequate for characterization of the plume chemistry and for differentiation of plume waters from native ground water. At a minimum, sodium (a major component of the salt in the materials disposed of), iron and sulfate (both useful in determining the origin of waters) should be analyzed for. Other metals or compounds may be useful in 'fingerprinting' the plume; chemical characterization of the materials disposed of would indicate which species are most appropriate.

The number of monitoring wells included in each chemical sampling and water-level measurement survey is much smaller than the total number of monitoring wells available for sampling. As a result, comparisons of data obtained during different sampling and measurement surveys are fraught with uncertainties. Use of these datasets for evaluation of the true extent of the problem and for prediction purposes is, therefore, not recommended; rather, complete surveys should be undertaken for this purpose.

The data obtained on the thickness, permeability, and hydraulic properties of the stratum referred to as the 'principal clay layer' or 'Kt4' are inadequate to conclude that it is continuous and nonleaky. The thickness of this stratum has been shown to vary considerably; most recently, a pump test that was conducted indicated that only a thin portion of the previously defined stratum may have such properties (the remaining portion was redefined as consisting of a permeable sand lense and a leaky clay substratum). The permeability estimates for the stratum were obtained by laboratory tests (permeameters) of small cores retrieved by use of split-spoon samplers; it is widely known that these methods routinely underestimate the true permeability of clay samples by several orders of magnitude. Hence, the use of such estimates for calculation of the potential time of travel through clay deposits is not to be viewed without question; the projections derived may vastly overstate potential travel times.

4.4.3. Validity of Interpretations

The presentations of water-level/piezometric countour maps by the plant operators (Olin Corporation, 1984) that were submitted to the governing agencies do not offer an objective assessment and differ from presentations prepared by their consultants using the same data (in an appendix to the 1984 report). Water-level values at some wells lie substantially outside contours which should enclose them (Figure 4.15), and none of the values have been adjusted to account for fluid density effects. These interpretive errors may result from incompetence or from purposeful shading of the data (more on this kind of behavior below) to generate support for the plant operators' assertion that the upper flow system consistently exhibits a riverward direction of flow.

For example, the effect of correctly (re-)positioning the piezometric contours for the 'alluvium aquifer', is to cause a southerly shift in the predominant direction of flow (Figure 4.16). Note that this causes the appearance of a ground-water mound beneath the active waste disposal areas ('E3' and 'E4') and southerly and westerly shifts in the predominant directions of flow indicated by the contours.

The obvious criticism that might be levelled at Figure 4.16 is that the existance of a ground-water mound is postulated from a single anomolous data point. The inclusion of additional data, however, strengthens this presentation (Figure 4.17). As may be noted in Figure

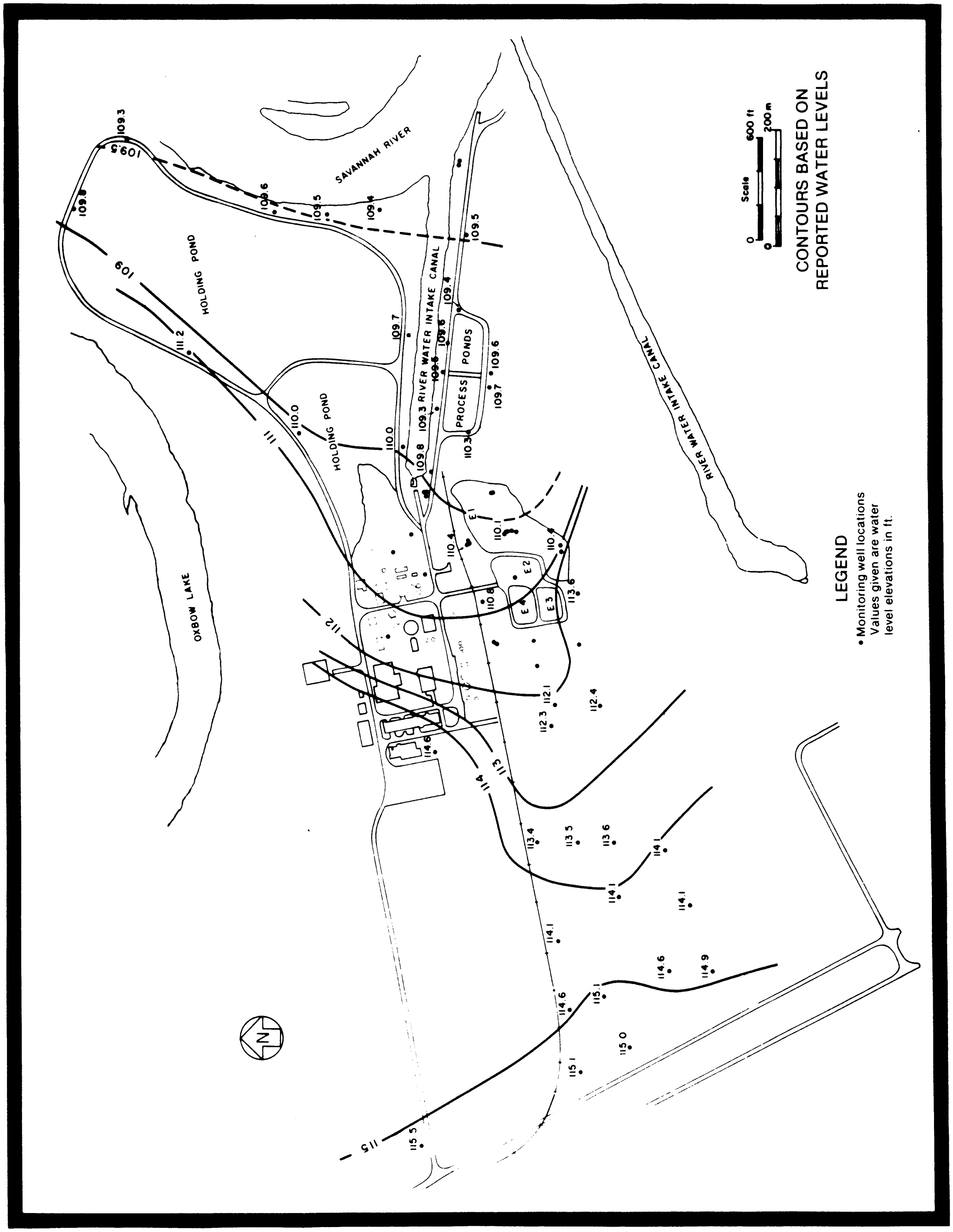


Figure 4.15. Water Level Contours as Reported

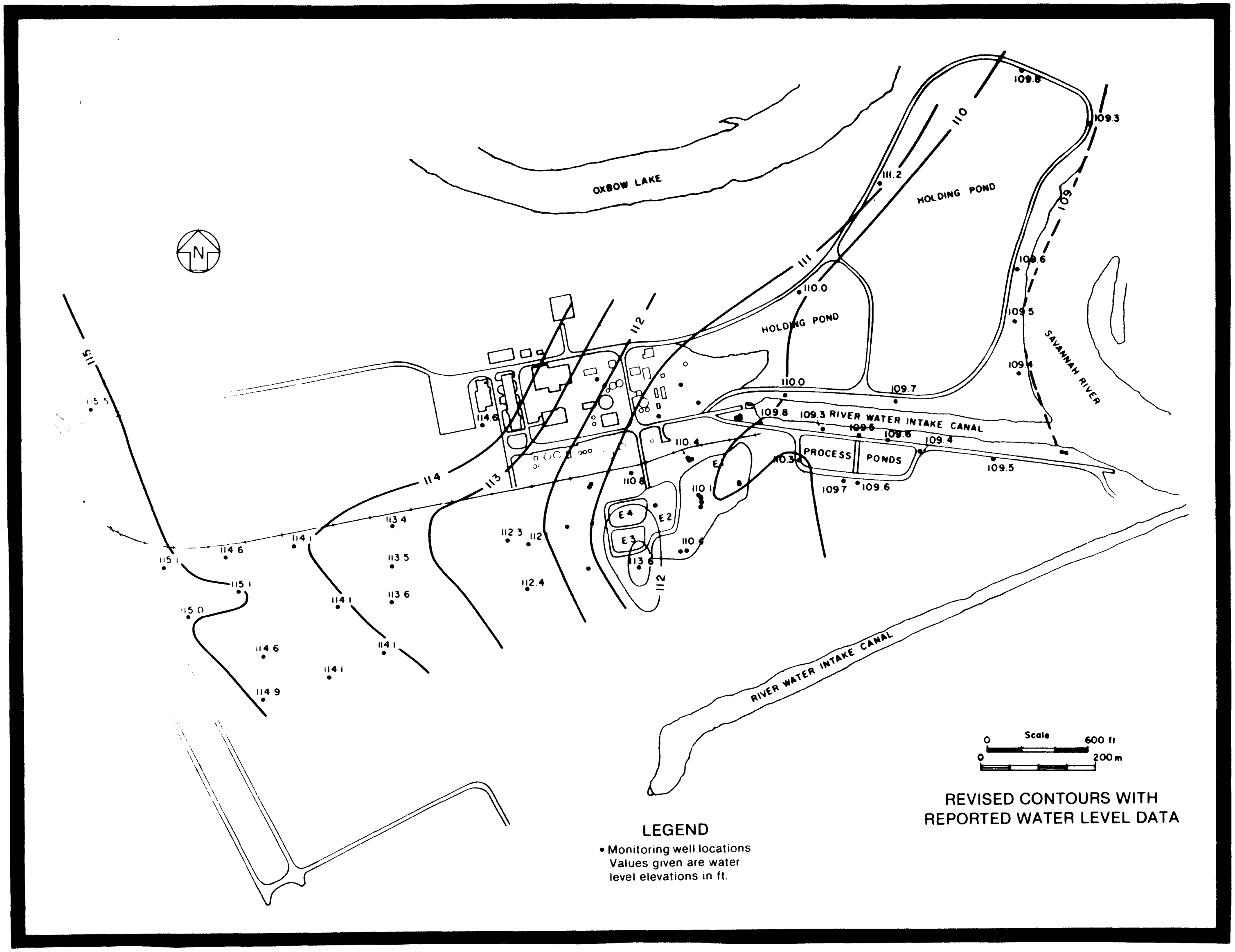


Figure 4.16. Repositioned Water Level Contours Using Same Data

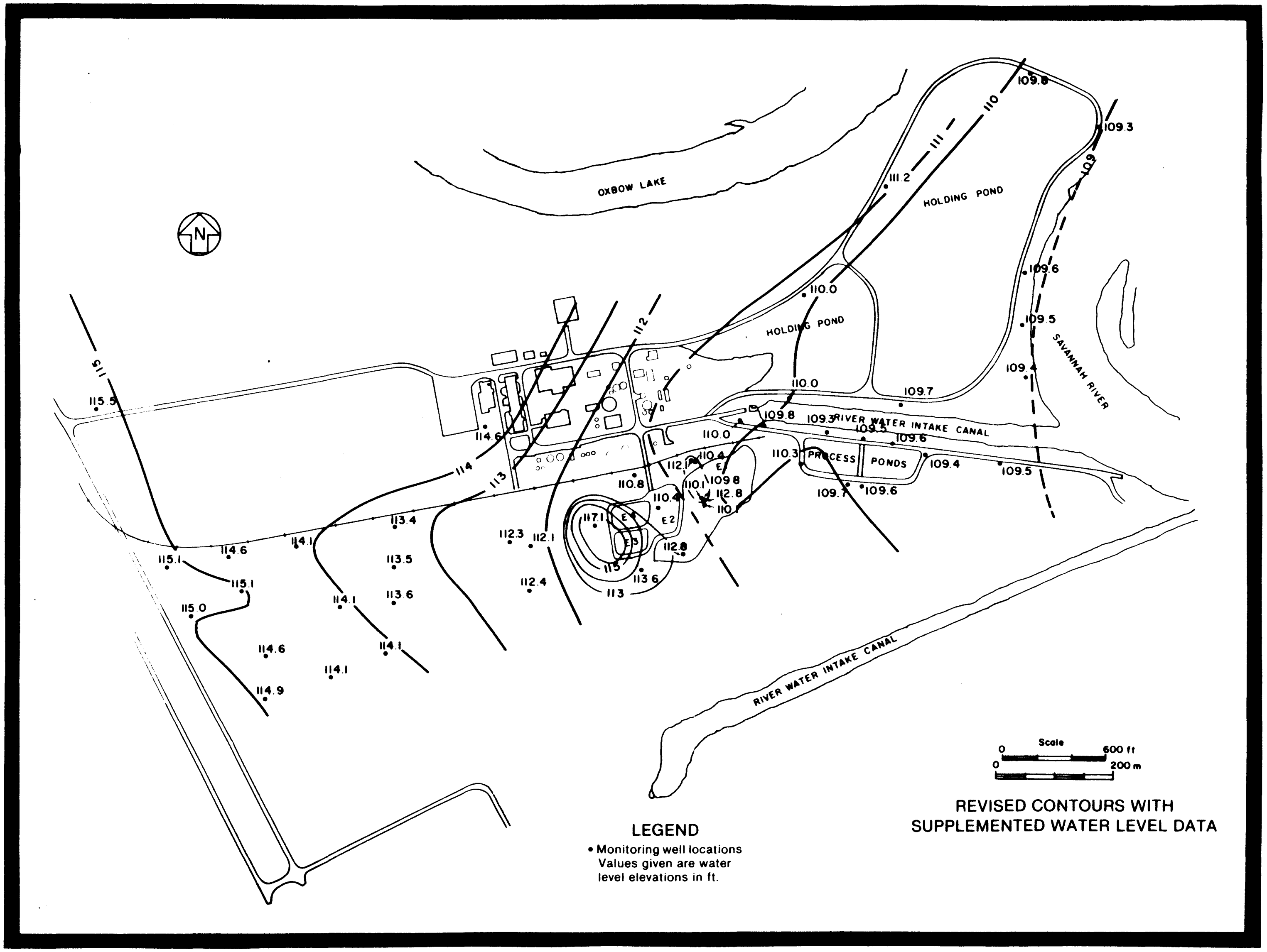


Figure 4.17. Water Level Contours Using All Available Data

4.14, there are far more monitoring wells in the alluvium aquifer than were shown with data values in Figure 4.15; this is particularly true in the vicinity of the waste disposal areas. One might suppose that the missing data values were the result of a limited data collection effort; but an examination of Appendix C of the 1984 report reveals that most of the missing data values were actually obtained during this effort. Examination of the well logs in Appendix D of the same report indicates that the monitoring wells with missing data values do indeed tap the alluvium aquifer (exception: Well no. 06). Hence, there is no apparent reason for exclusion of the missing data values and no apparent reason not to present the unadjusted water-level data as shown in Figure 4.17.

The effect of adjusting the measured water-level values to account for fluid density differences can not be determined without additional information regarding the salt content of waters at each well. As an example of the magnitude of effect density differences can have, water having a salt content of 100,000 ppm as sodium chloride (or chloride level at approx. 65,000 ppm) has a specific gravity (its density relative to pure water) of 1.0734. For a 100-foot thick stratum saturated with this quality of water, the 'freshwater-equivalent head' is 107.34 feet or 7.34 feet greater than if the stratum contained pure water. Appendix A of the 1984 report gives chloride values for most of the alluvium aquifer monitoring wells during the March 1983 field work. Using those data, and assuming an average saturated thickness of 33 feet for the alluvium aquifer (based on the well logs), the water-level data were adjusted and replotted as freshwater-equivalent values in Figure 4.18. As can be seen there, the effect is to strengthen the argument for the existence of a ground-water mound beneath the waste disposal areas.

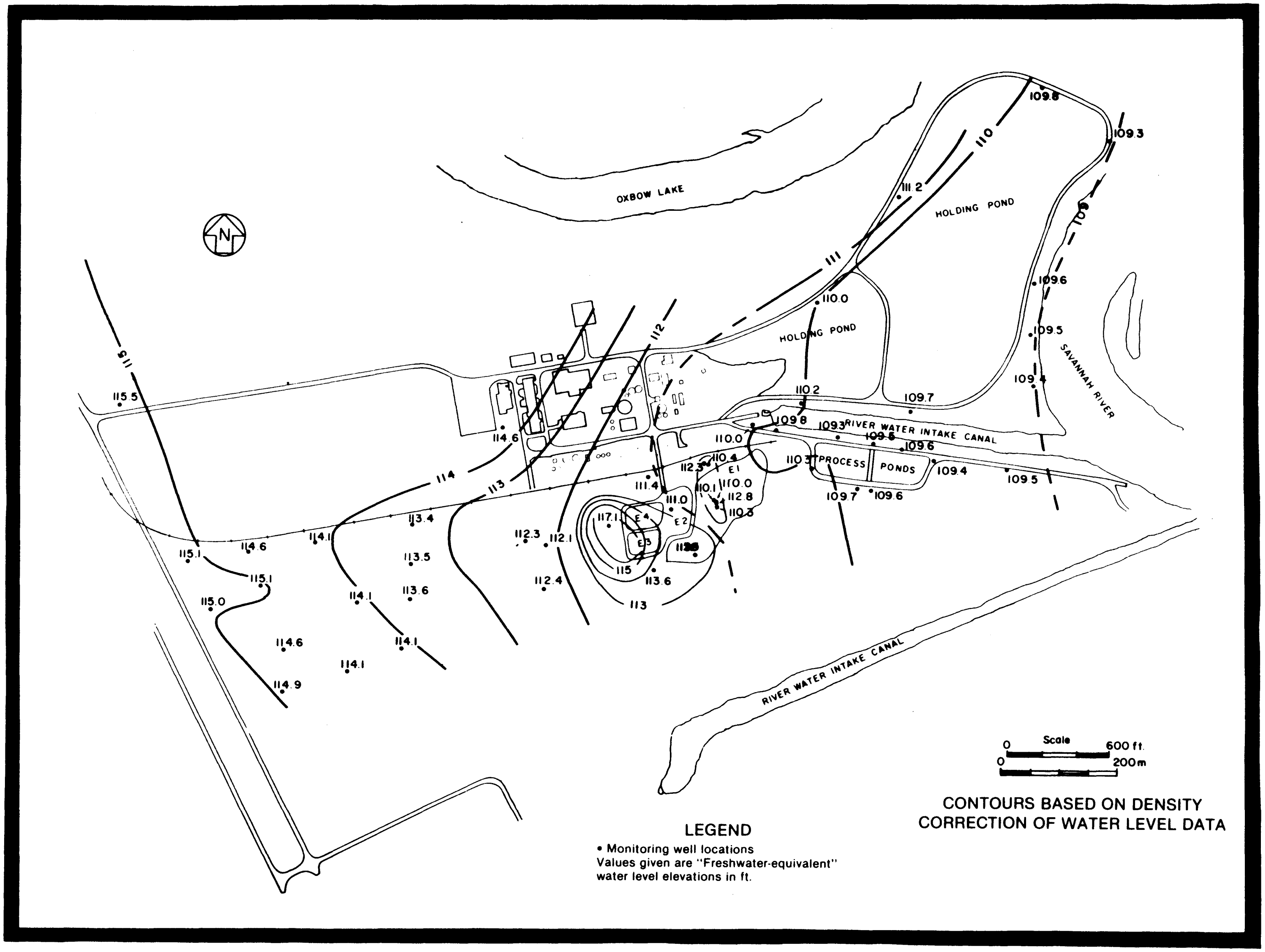
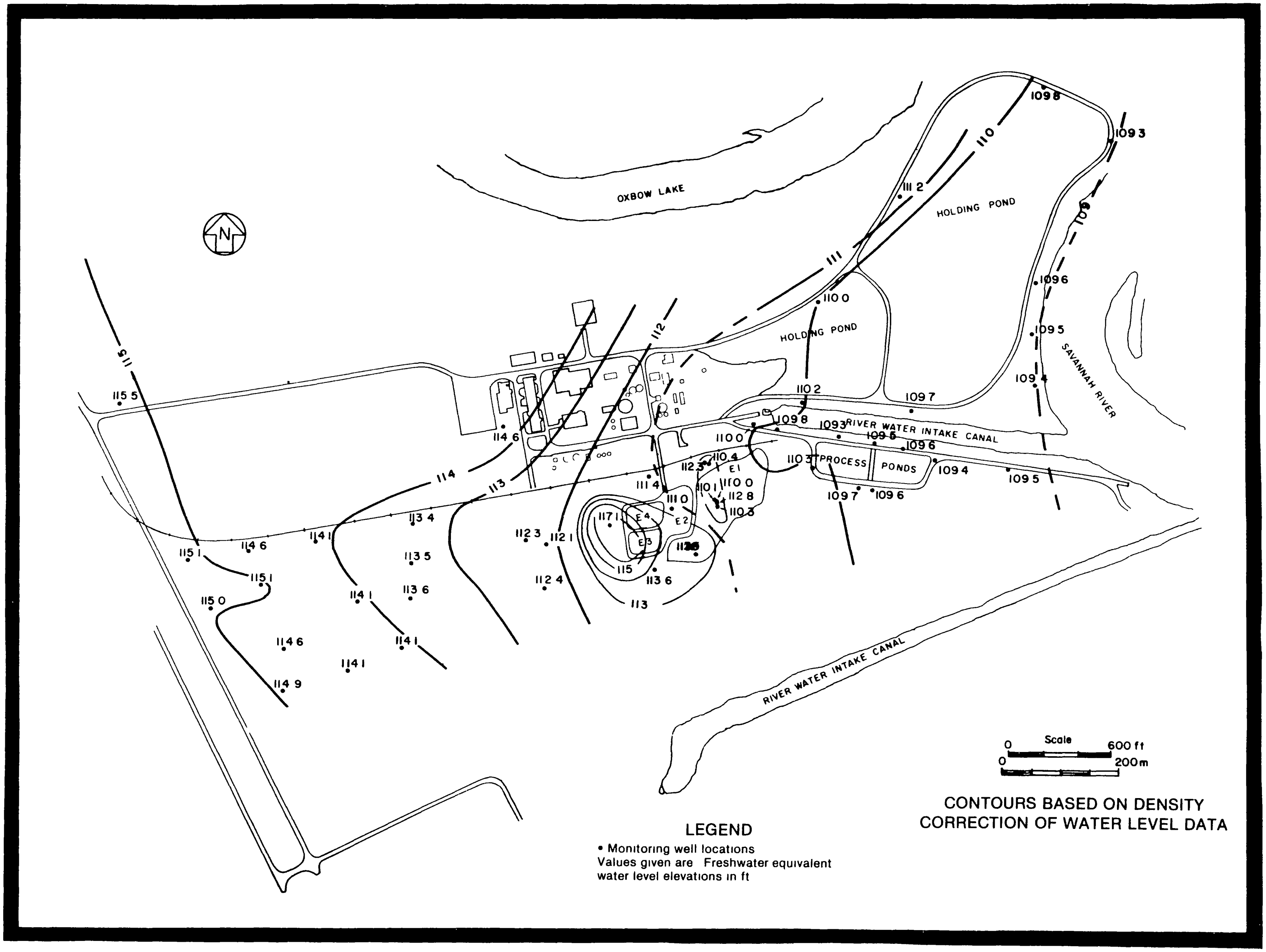


Figure 4.18. Density-Corrected Water Level Contours



CONTOURS BASED ON DENSITY CORRECTION OF WATER LEVEL DATA

LEGEND
• Monitoring well locations
Values given are Freshwater equivalent water level elevations in ft

Figure 4.18. Density-Corrected Water Level Contours

A final comment with regard to the water-level data presented in the 1984 report relates to the large discrepancies in water-levels between adjacent wells. For example, wells 13, 14, 15, and 15A are clustered together in abandoned solid waste disposal area 'E1' and show a range of values from 110 feet to 112.8 feet; wells 11 and 12 are side by side and exhibit a similar range. Since there is no obvious hydrogeologic reason for such discrepancies, the question of which wells are valid sampling/measuring points really should be addressed in an unambiguous manner. Note that the resolution of this issue is required before complete definition of the water-level/piezometric contours and ground-water flow directions can be given (these uncertainties are indicated in Figures 4.17 and 4.18 by dashed contours).

The lack of a sufficient number of deep monitoring wells also makes it difficult to accept the structural/stratigraphic representation that the plant operators' reports present for this complex subsurface system. A description of the stratigraphy is provided in the text of the plant operators' 1984 report. The text description is accompanied by a series of cross-sectional maps and is fairly explicit, especially regarding the supposedly continuous aquitard ('Kt4') overlying the deepest aquifer ('Kt5') that is heavily used for local municipal water supplies (Olin Corporation, 1984, pp.23-24):

> "Of the ten borings on the site that have gone below the 180 foot depth all have encountered additional clay strata. Two of the clay strata are found in all boreholes. These two clay layers with an intervening sand make up the Kt4 unit as used in this report. The upper of these is referred to [by one of their consultants] as the 'principal clay layer'. Below the Kt4 unit is Kt5, the 'principal' aquifer...[Both of their consultants] are in agreement that the main aquifer in the area...is isolated from all other aquifers by a 'Principal Clay Layer,' Kt4."

The drillers logs given in Appendix D of the 1984 report, however, do not support such definitive statements. Figures 4.19 and 4.20 present examples of the drillers logs; both were used in the cross-sectional maps presented in the main body of the 1984 report. In Figure 4.19 it can be seen that the driller reports only one clay encountered at the depth appropriate for 'Kt4' in well Z24, a fairly minor occurrence (approx. 5 ft. thick) at a depth of about 170 ft. In the cross-sectional map in which Z24 is used in the main body of the report, however, a second clay layer appears (approx. 10 ft. thick, at a depth of about 190 ft.); this bogus clay layer is clearly outside the bounds of artistic license.

In Figure 4.20 it can be seen that the driller reports the occurrence of a total of four clay layers at the appropriate depth for 'Kt4' and deeper in well Z25. In the cross-sectional map in which Z25 is used in the main body of the 1984 report, the two deepest clay layers (at 205 ft. and 220 ft. depths) are labelled 'Kt4' and the other two (at 160 ft. and 180 ft. depths) are shown as discontinuous lenses in the 'Kt3' aquifer overlying 'Kt4'.

These kinds of presentations seem to be stretching the facts in a blatantly self-serving effort; the plant operators seem to want desperately to show that the major production aquifer is isolated from potential contamination. Geologic principles alone suggest that, if clay strata are extremely variable in thickness and stratigraphic position (especially for thin strata), it is entirely likely that the clay strata are discontinuous (hence the major differences in wells Z24 and Z25). Common experiences of hydrogeologists working in this kind of

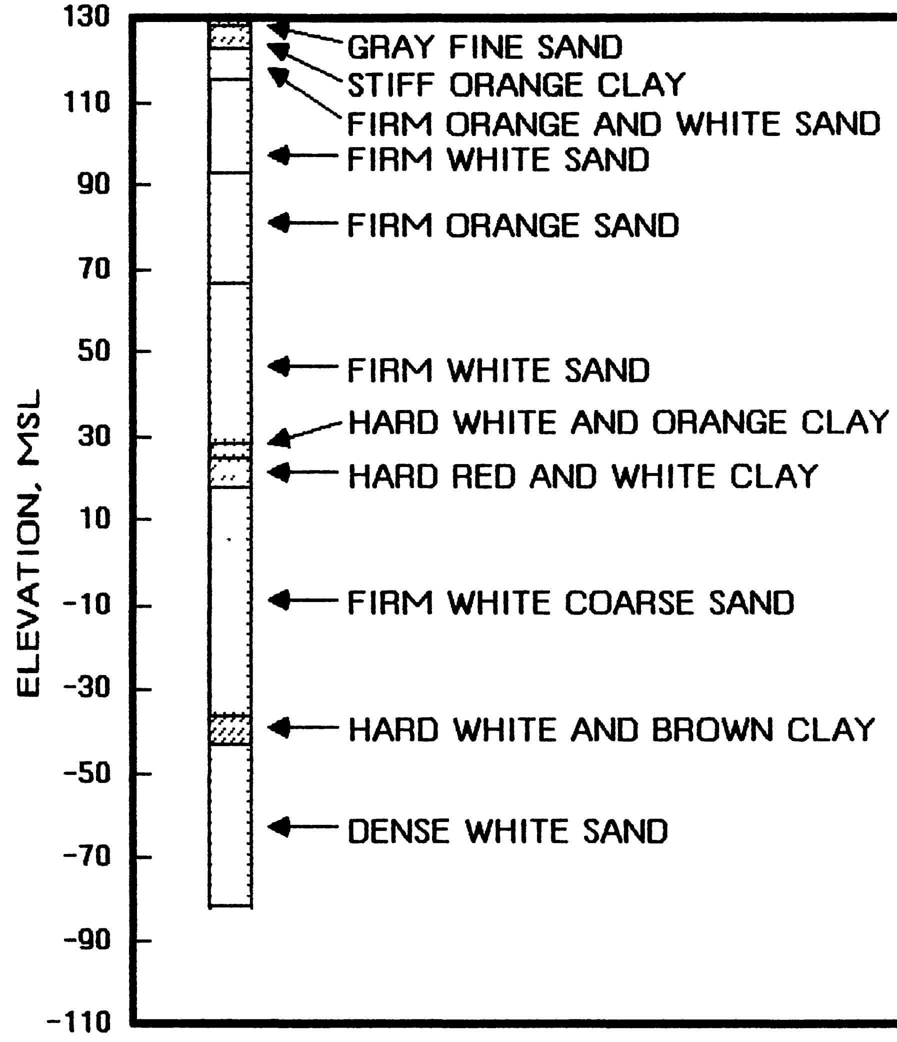


Figure 4.19 Driller's Log for Monitoring Well Z-24

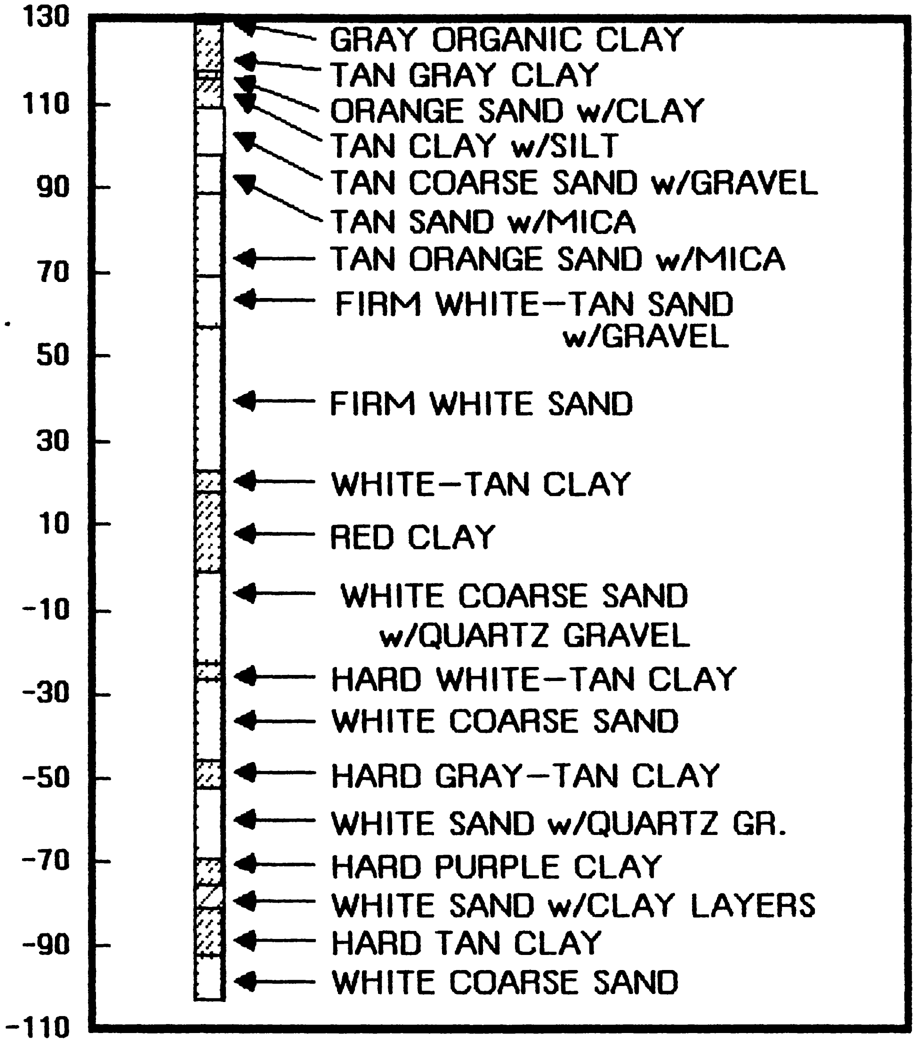


Figure 4.20 Driller's Log for Monitoring Well Z-25

depositional environment also show that such strata are rarely continuous over more than a few hundred feet of lateral extent.

The interpretations offered in the 1984 report regarding the potential for movement of fluids from the uppermost to deeper strata do not adequately consider all of the data. In particular, no mention is made of the significant downward hydraulic gradient that is apparent by comparison of the water-level elevations of pairs of shallow and deep wells. These downward vertical gradients were clearly shown in a consultant's report to the plant operators (an appendix in the 1984 report), represented by arrows drawn adjacent to specific paired values of water levels. Incorporation of this information, and the findings of traces of contamination in 'Kt5', raises serious questions about the contention that 'Kt4' is everywhere continuous and non-leaky.

4.4.4. Feasibility of Proposed Remediation

The basis of the remediation efforts proposed in the 1984 report is faulty due to the errors and omissions noted above as well as the quality of the numerical modeling performed by the plant operator's consultants. Contrary to their claims, the numerical modeling resulted in poor agreement between the predicted and observed drawdowns. For example, the model predicted 0.5 feet drawdown at observation well X13, located 250 feet from pumping well G1, whereas the observed drawdown was actually 1.5 feet (200% greater than predicted). In some respects this degree of error is expected; the aquifer coefficients that were input to the model were arrived at by incorrect analysis of the pump test results (i.e., the line fitted to the semilog drawdown plot for well GM-2 was heavily weighted towards a single observation not lying on the line suggested by the remaining observations).

The rate and duration of pumpage suggested by the plant operators for remediation are not well founded. Their consultants have incorrectly stated that 'doubling the 100 or 40 gpm pump rates will not significantly reduce the time necessary to draw the outer edge of the plume to the wells.' Actually, the velocities of ground-water flow are directly proportional to the pumping rate for semiconfined and confined aquifers, such as present in this case; doubling the pumping rate will double the velocity. Examination of the governing equations, as done here in preceding chapters, shows this to be the case; drawdowns caused by pumpage are also directly proportional to the pumping rate for semiconfined and confined aquifers.

The proposed remediation in the 1984 report presumes that little more than one flushing of the aquifer is necessary to capture and remove the contaminants, based on laboratory experiments with brine solutions that indicated that the brine would be displaced by 'plug flow'. This presumption belies a lack of appreciation of the difference between a controllable laboratory experiment and the severe complications of natural settings, as discussed in preceding chapters.

The sampling protocols proposed by the plant operators for use in remediation efforts were improper and inadequate. Separate samples for chloride and mercury should be taken at each monitoring point; the sample for chloride determination would need no special preservatives, but the sample for mercury would need to be preserved immediately in accordance with official recommendations (EPA, 1979). To prevent inadvertant contamination of successive samples, the sampling devices should be thoroughly flushed with acidified water (pH < 2, with HNO_3); one liter of acidified water would be adequate for flushing peristaltic pumps, but a much greater volume would be necessary for submersible pumps. The latter could be accomplished by submersion of the pump in a large plastic container (i.e., 40 gallon Rubbermaid™) filled with acidified water; the pump would be operated for several minutes while discharging into the same container (recirculation flushing) and would be subsequently immersed in a second such container filled with pure water for rinsing by similar technique.

Given the uncertainties that yet exist regarding characterization of the spatial distribution of the contaminants at this site, it is premature to judge the propriety and adequacy of designs for a compliance monitoring network. There are, however, some precepts that should be adhered to. The most important of these is that the locations of the monitoring wells to be used for compliance, and their sampling schedules, be sufficiently well chosen to provide an unambiguous database. This means, for example, that the database be able to demonstrate the effects of remediation efforts without the 'assistance' of a numerical model or other data-smoothing/interpretive tools. The rationale here is one of trying to avoid a situation in which a subjective tool becomes a replacement for hard field data.

A second major precept is that compliance be demonstrated by physical as well as chemical methods, due to the great variations in chemical parameters that would be expected. Specifically, the maintenance of inward hydraulic gradients (both vertically and horizontally) from the plume boundary toward the extraction wells should be required. Finally, another crucial precept is that source control efforts (i.e., selective removal of soil hot-spots and emplacement of a multilayer impermeable cap over the residuals) be subjected to post-installation monitoring measures that are capable of ensuring their continuing effectiveness (e.g., monitoring the movement of moisture beneath the cap by the use of neutron probes).

4.5 Case History No.4

4.5.1 An Archetypical Superfund Site

Similar experience with special uses of geotechnical methods and state-of-the-art research findings occurred at the 20-acre Chem-Dyne solvent reprocessing site in Hamilton, Ohio (Figure 4.21). In excess of 250 chemical waste generators had sent drummed or bulk wastes to the site during its operational lifetime (1974-1980). Poor waste handling practices, such as purposeful onsite spillage of a wide variety of industrial chemicals and solvents, direct discharge of liquid wastes to a stormwater drain beneath the site, and mixing of incompatible wastes were engaged in routinely at Chem-Dyne. These practices caused extensive soil and ground-water contamination, massive fish kills in the Great Miami River, and major onsite fires and explosions, respectively.

The stockpiling of liquid and solid wastes resulted in a long-term threat to the environment. More than 50,000 drums of hazardous waste had been stored at the site at its peak of operations (CH_2M-Hill, 1984a). The drums were improperly stacked in tiers five and six drums high, causing the lowest tiered drums to buckle and corrode. When the author viewed the

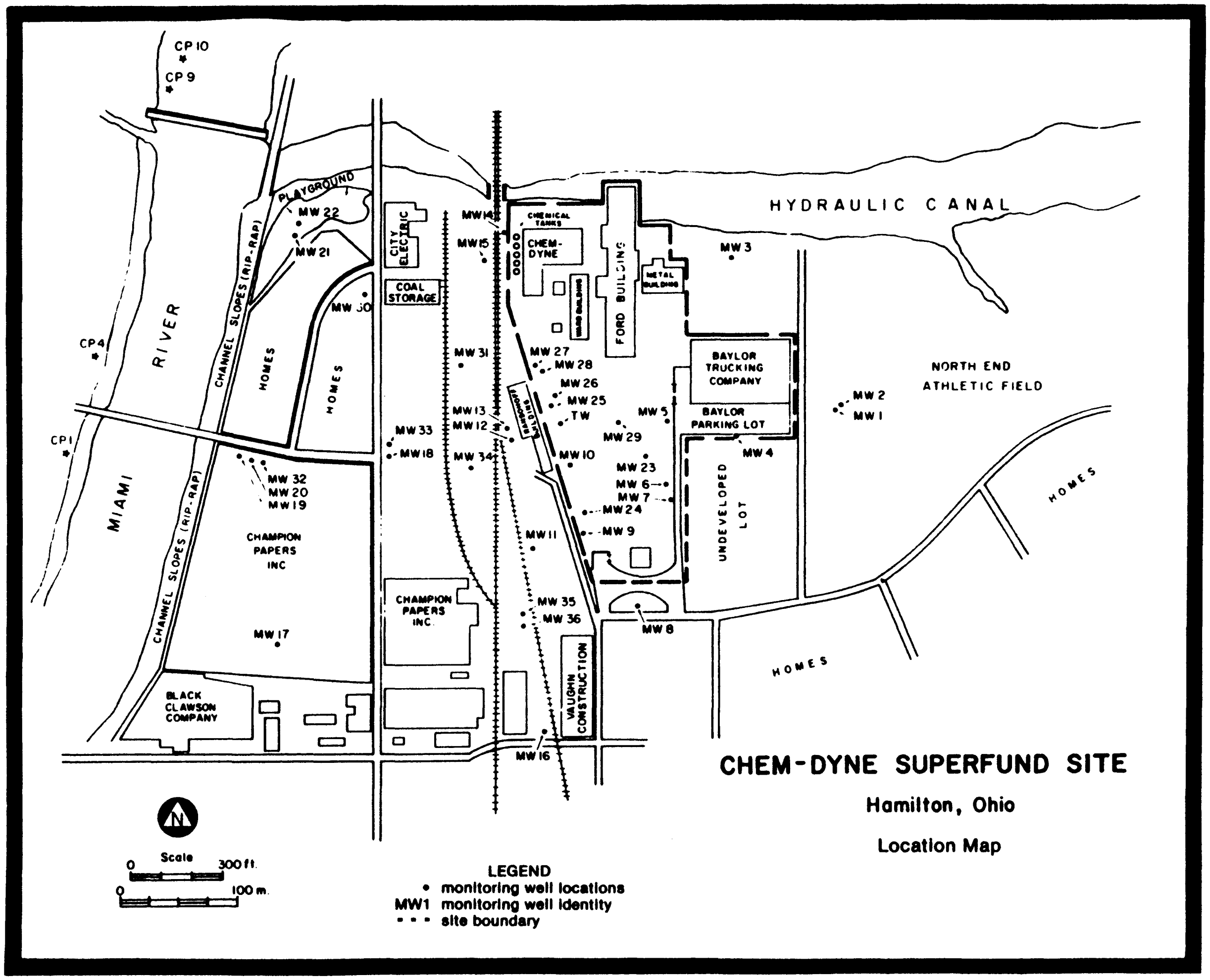


Figure 4.21. Monitoring Well Location Map for Case History No.4

site in March, 1982, more than 20,000 drums still remained with more than 8,500 of these so badly corroded that they could not be identified. A number of bulk chemical storage tanks had also been abandoned onsite. Visual observations indicated that raw chemical salts and oils had been poured out upon the sand-and-gravel ground surface.

4.5.2 The First Investigation

The seriousness of the ground-water contamination problem became evident during the initial site survey (1980-1981), which included the construction and sampling of over twenty shallow monitoring wells (Ecology and Environment, 1981). The initial survey indicated that the contaminant problem was much more limited than was later shown to be the case (Roy F. Weston Inc., 1983, CH_2M-Hill, 1984a). A good portion of the improvement in delineating the plume was brought about by an improved understanding of the natural processes controlling transport of contaminants at the site.

The initial site survey indicated that ground-water flow was generally to the west of the site, toward the Great Miami River, but that a shallow trough paralleled the river itself as a result of weak and temporary stream influences. The study concluded that contaminants already in the aquifer would be discharged into the river and need not be removed as a result of this natural cleansing mechanism (Ecology and Environment, 1981). That study also concluded that the source was limited to highly contaminated surface soils, and that removal of the uppermost three feet of the soil would essentially eliminate the source of contaminants.

That conclusion was, however, based on faulty soil sampling procedures. The soil samples that were taken were not preserved in air-tight containers, so that most of the volatile organic chemicals leaked out prior to analysis. That the uppermost soil samples showed high volatile organic levels is probably explained by the co-occurrence of viscous oils and other organic chemicals that may have served to entrap the volatiles. The more viscous and highly retarded chemicals did not migrate far enough into the vertical profile to exert a similar influence on samples collected at depths greater than a few feet.

4.5.3 Additional Site Investigations

Subsequent studies of the site corrected these misinterpretations by producing data from proper soil samplings and by incorporating much more detailed characteriztions of the fluvial sediments and the natural flow system. In those studies vertical profile characterizations were obtained from each new borehole drilled, by continuous split-spoon samples of subsurface solids. The split-spoon samples helped to confirm the general locations of intefingered clay lenses and clearly showed the high degree of heterogeneity of the sediments (Figures 4.22 and 4.23). For example, a dense clay lense can be found at similar elevations (at 570 to 580 ft.MSL) along the valley axis (Figure 4.22), but is found only intermittently perpendicular to the valley axis (Figure 4.23). This should be expected by hydrogeologic intuition, since rivers dissect deposits that were laid down in low energy periods everytime they flood, and with the natural channel changes they undergo (e.g., meandering) as they mature. What it means when those same sediments become buried over

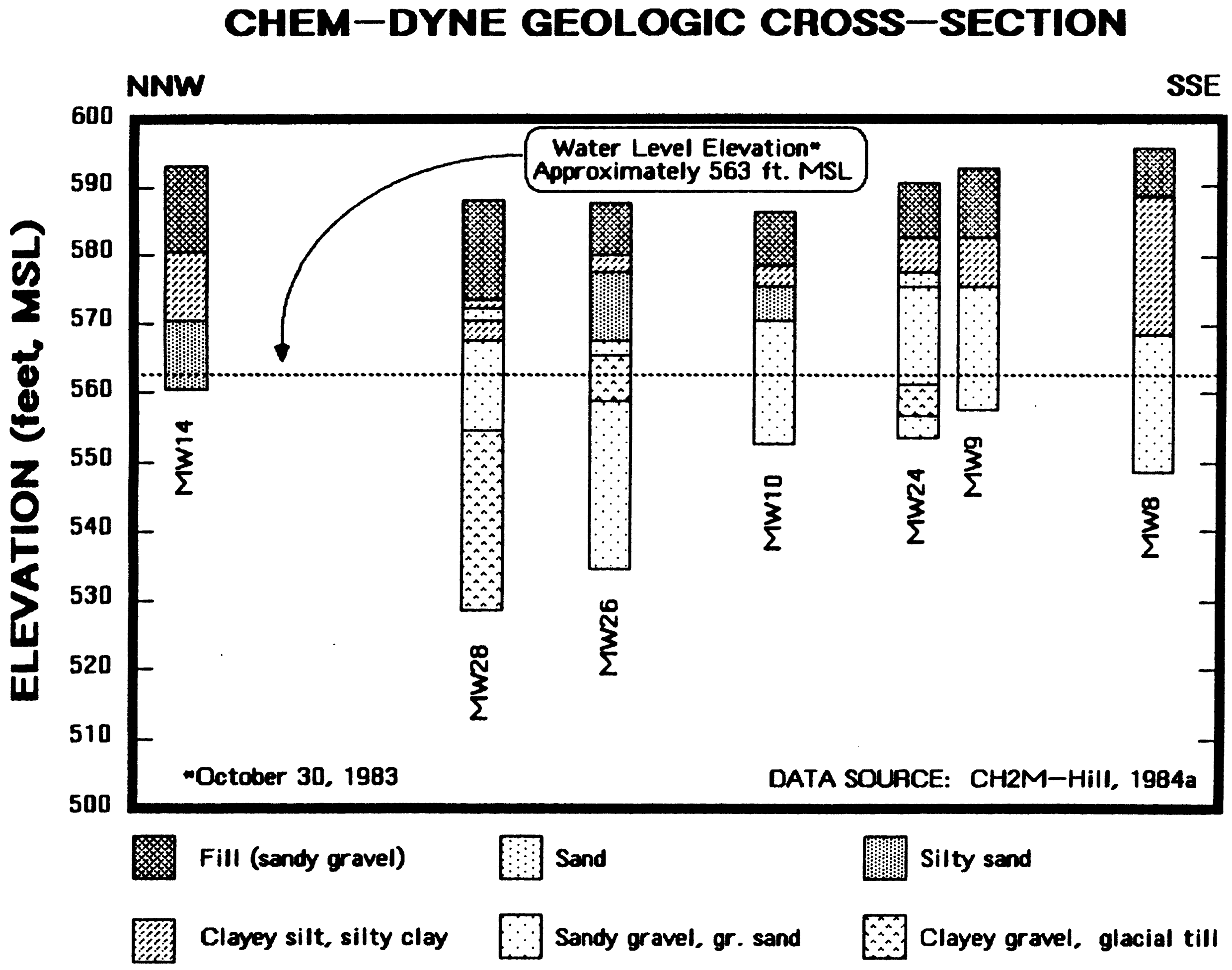


Figure 4.22. NNW-SSE Geologic Cross-Section at Chem-Dyne Site

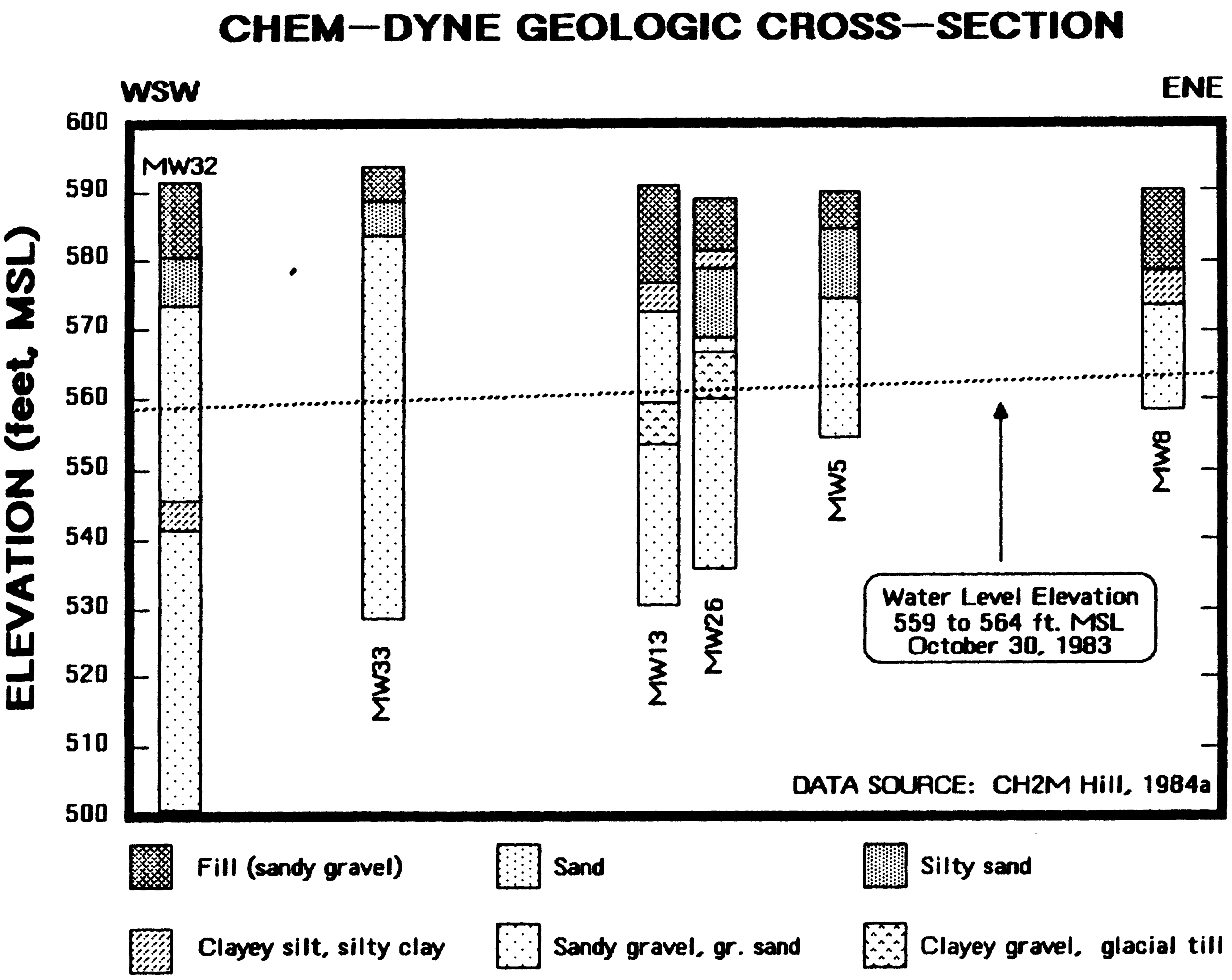


Figure 4.23. WSW-ENE Geologic Cross-Section at Chem-Dyne Site

the years, is that lenticular clay bodies of very limited extent accrue and create structurally anisotropic conditions. The major axis of anisotropy parallels the average downstream direction, which itself is usually in line with the valley axis.

Clusters of vertically-separated monitoring wells (Figure 4.24) were constructed during the latest investigations at Chem-Dyne at the author's insistence. While an extensive network of shallow wells confirmed earlier indications of general ground-water flow toward the river (Figure 4.25), the clusters of vertically separated wells revealed that <u>downward</u> gradients existed adjacent to the Great Miami River. Figures 4.26 and 4.27 show that these vertical gradients, which ranged 1-3 ft. drop over the 20 ft. vertical separation between the bottom of the shallow wells and the top of the screens in the deep wells (or about 0.100), are quite dramatic relative to the horizontal gradient across the contaminant plume (which averages about 0.001). This finding indicated that the migrating plume would not be discharged to the river, but would instead flow under the river.

The presence of major industrial wells on the other side of the river provided an explanation for the observed downward vertical gradients (normally, one would have expected the river to be gaining water from the aquifer at this point in the basin), and supported the conclusion that contaminants could not be discharged to the river from the aquifer. The plume would be drawn to greater depths in the aquifer by the locally severe downward gradient, but whether the industrial wells would actually capture the plume could not be determined. That determination would require careful evaluation of the hydrogeologic features beneath the

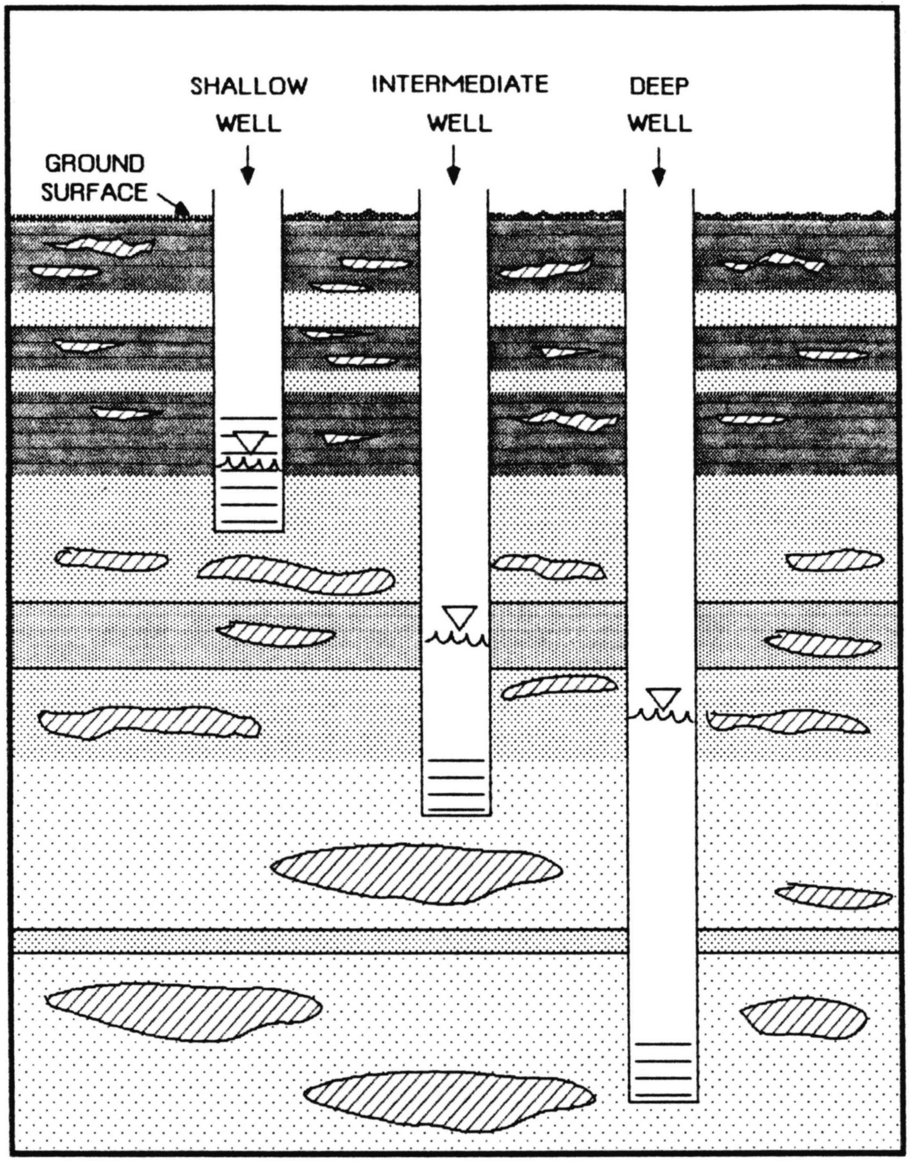


Figure 4.24. Conceptualization of Cluster of Vertically-Separated Monitoring Wells Installed in Chem-Dyne Site Studies

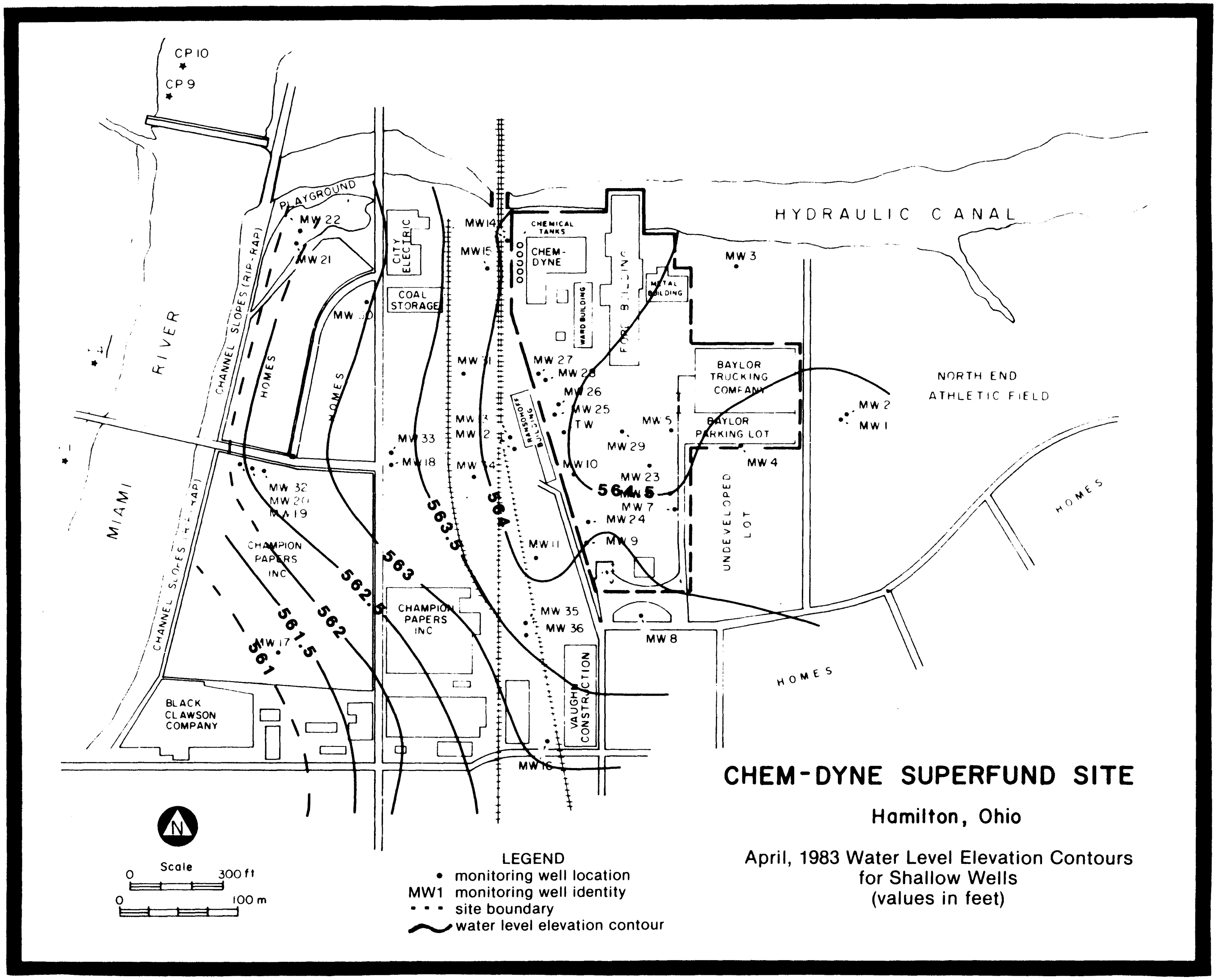


Figure 4.25. April, 1983 Water Level Elevation Contours at Chem-Dyne Site Using Shallow Well Data Only, in Feet MSL.

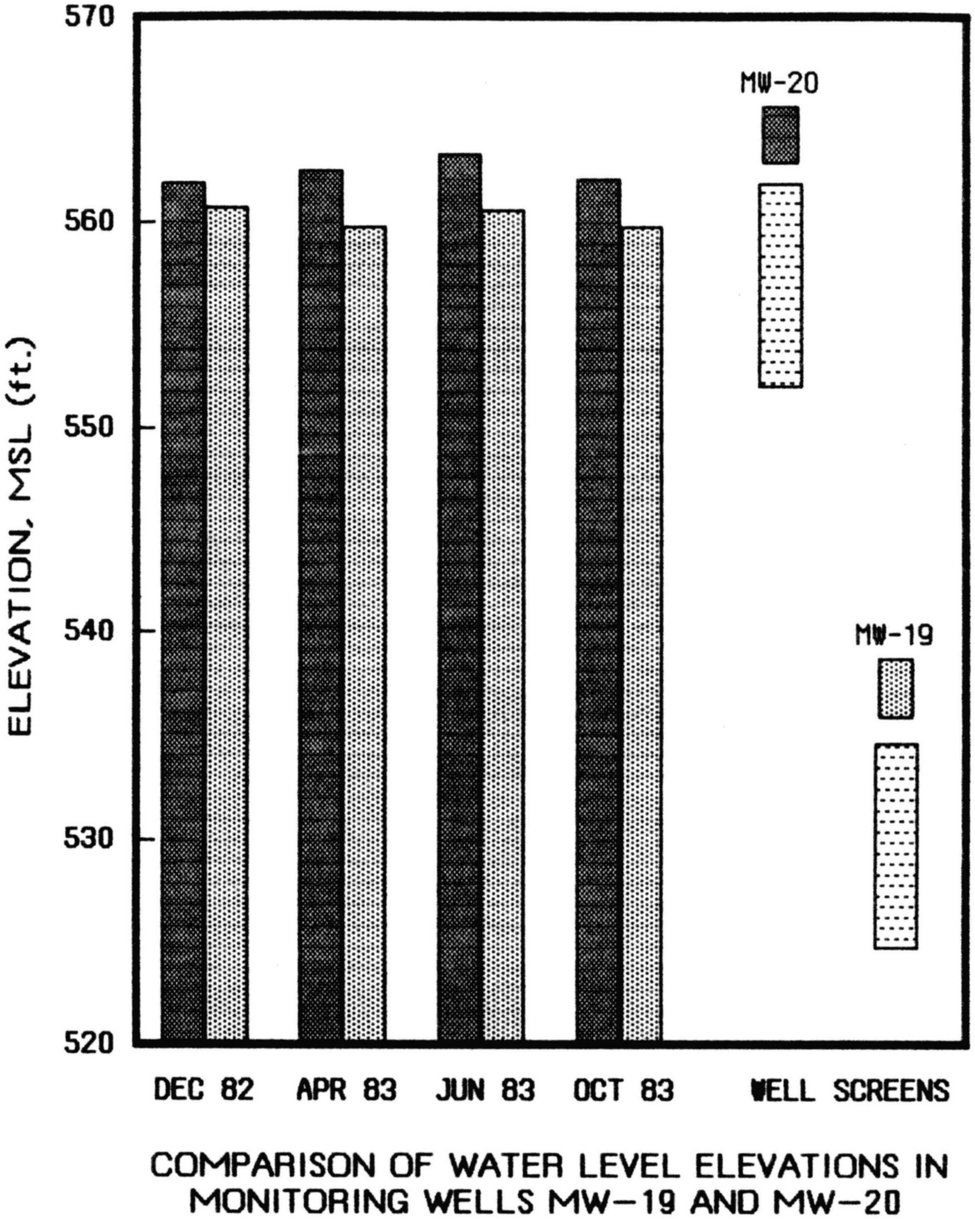


Figure 4.26. Water Levels in a Cluster of Two Vertically-Separated Monitoring Wells Adjacent to Great Miami River, and due West of Center of Chem-Dyne Site

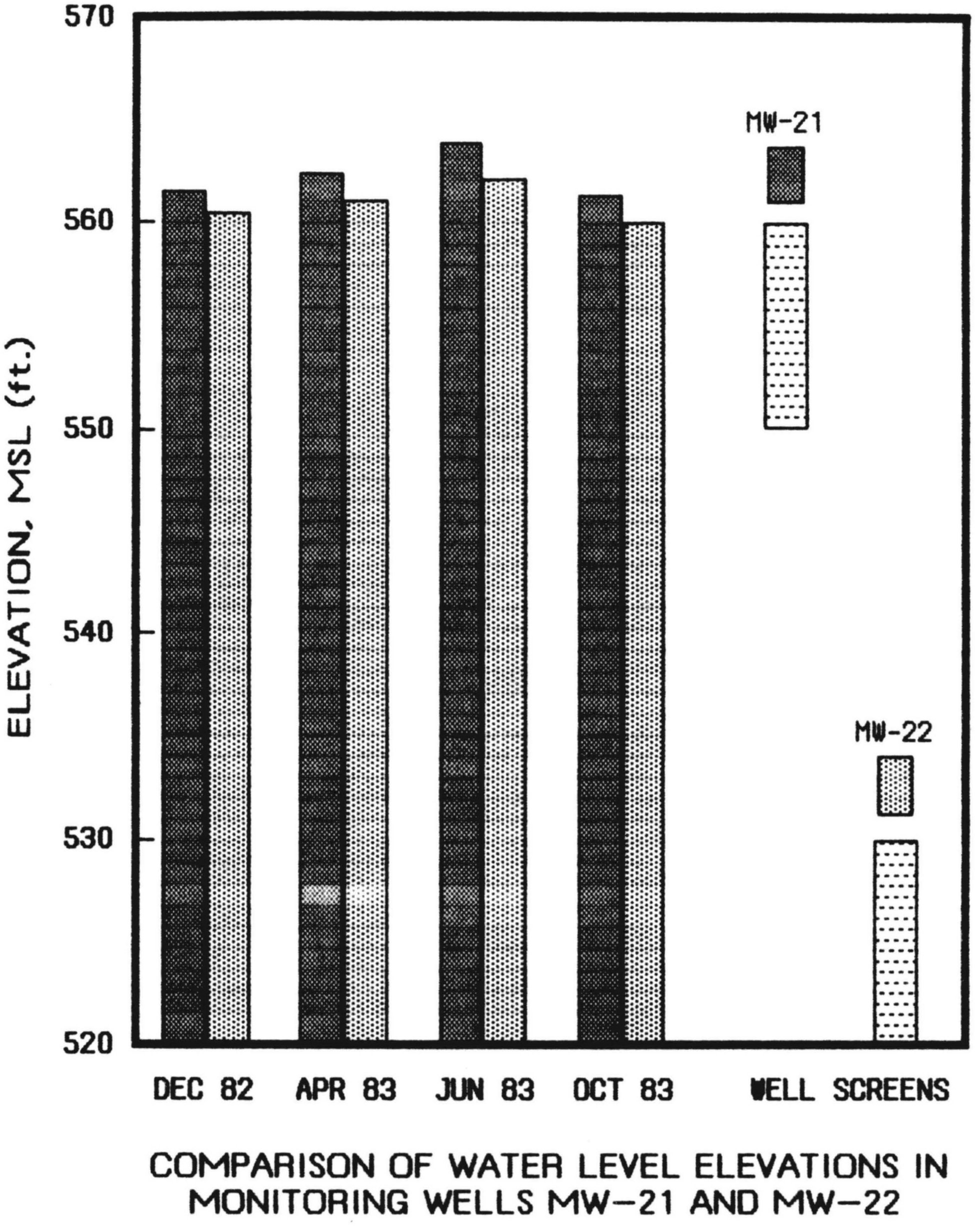


Figure 4.27. Water Levels in a Cluster of Two Vertically-Separated Monitoring Wells at the Confluence of Great Miami River and Ford Hydraulic Canal, due West of North Boundary of Chem-Dyne Site

river; something that has not been attempted because of the expense that woud be involved, and expectations that planned remedial actions would stop the plume before substantial encroachment could occur.

4.5.4 Hydrologic Complications

Unfortunately, the hydraulic effects that the river could exert on the aquifer were not well appreciated by the field crews taking routine measurements of water levels. As observed by the author late in the final study, during preparations for the pump test, the river stage variations cause as much as three feet of water level changes to occur at wells close to the river during a single day, though the effect is virtually negligible at wells much closer to the site. This sort of situation makes it crucial to obtain water levels at all wells within only a few hours; otherwise the sort of confused water level maps that are shown in Figures 4.28 to 4.30 will result. Those figures were prepared from the data logs of the field crews at Chem-Dyne, which indicate that measurements were often taken over a period of several days (CH_2M-Hill, 1984a).

4.5.5 The Pump Test

The author argued successfully for the performance of a major pump test so that the hydrogeologic characteristics of the heavily contaminated portion of the aquifer could be estimated. Given the projected costs of the test, including the drilling of a major well onsite, there was considerable resistence to conducting the test. The pump test was difficult to arrange technically, because the pumping well

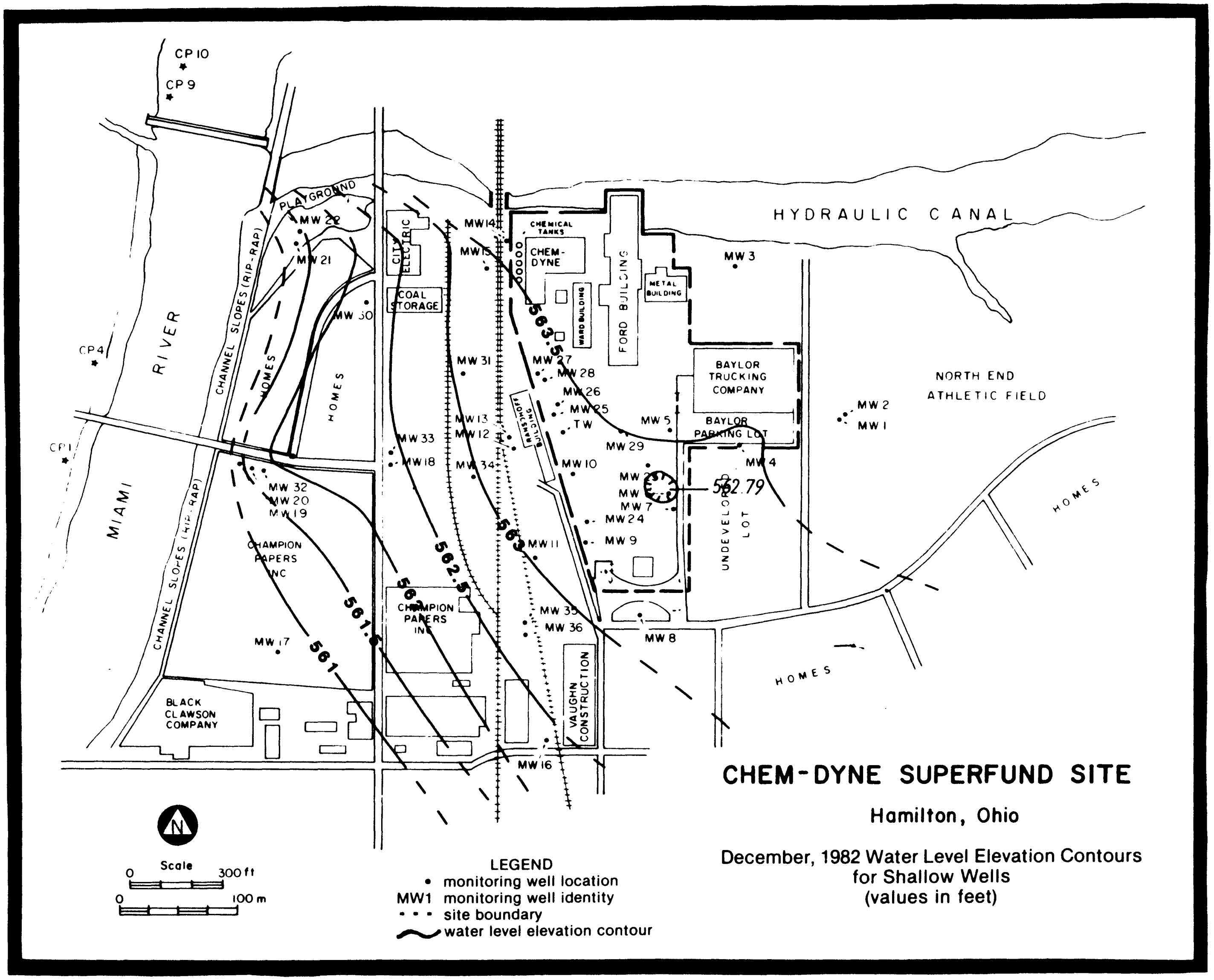


Figure 4.28. December, 1982 Water Level Elevation Contours at Chem-Dyne Site Using Shallow Well Data Only in Feet MSL

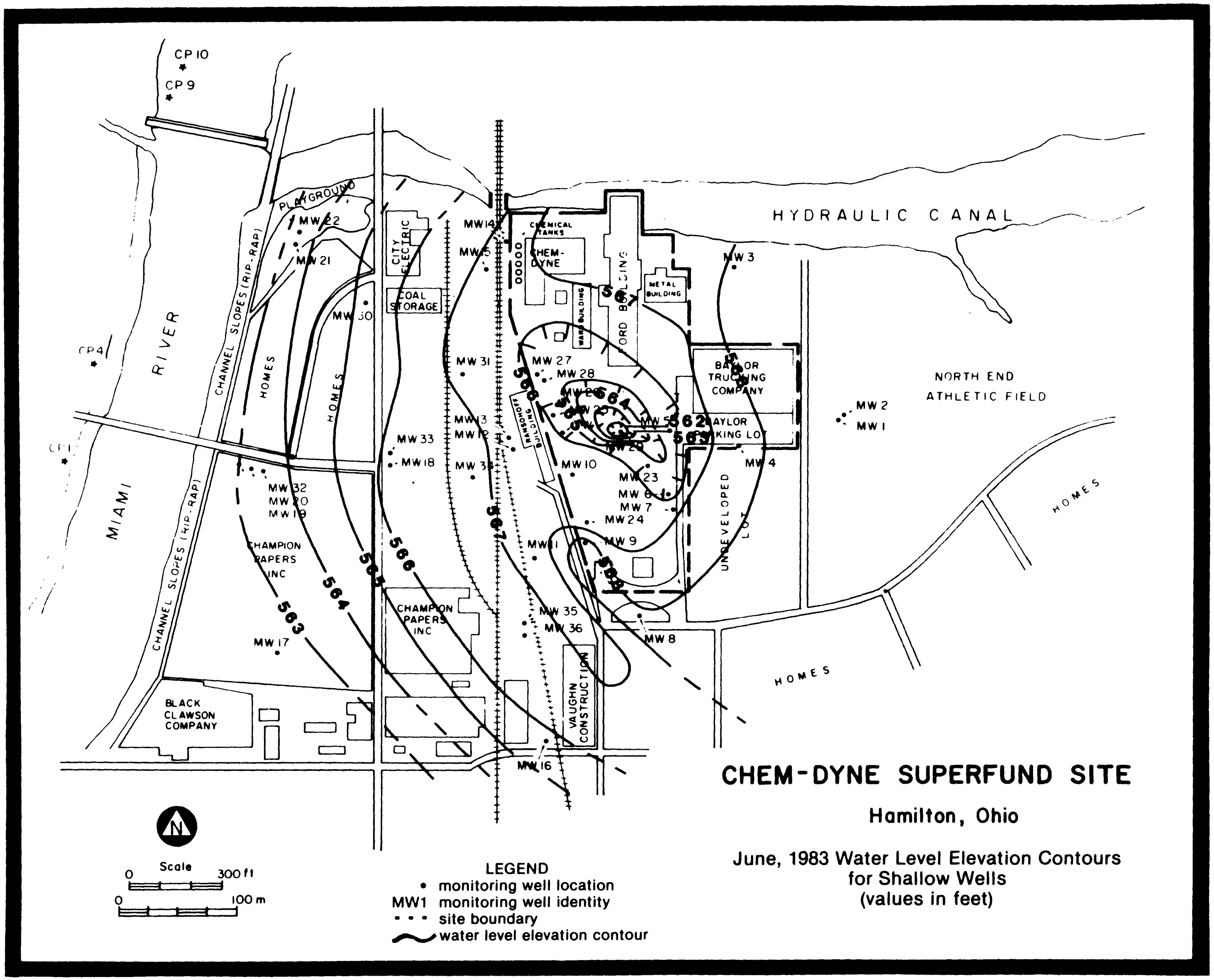


Figure 4.29. June, 1983 Water Level Elevation Contours at Chem-Dyne Site Using Shallow Well Data Only, in Feet MSL

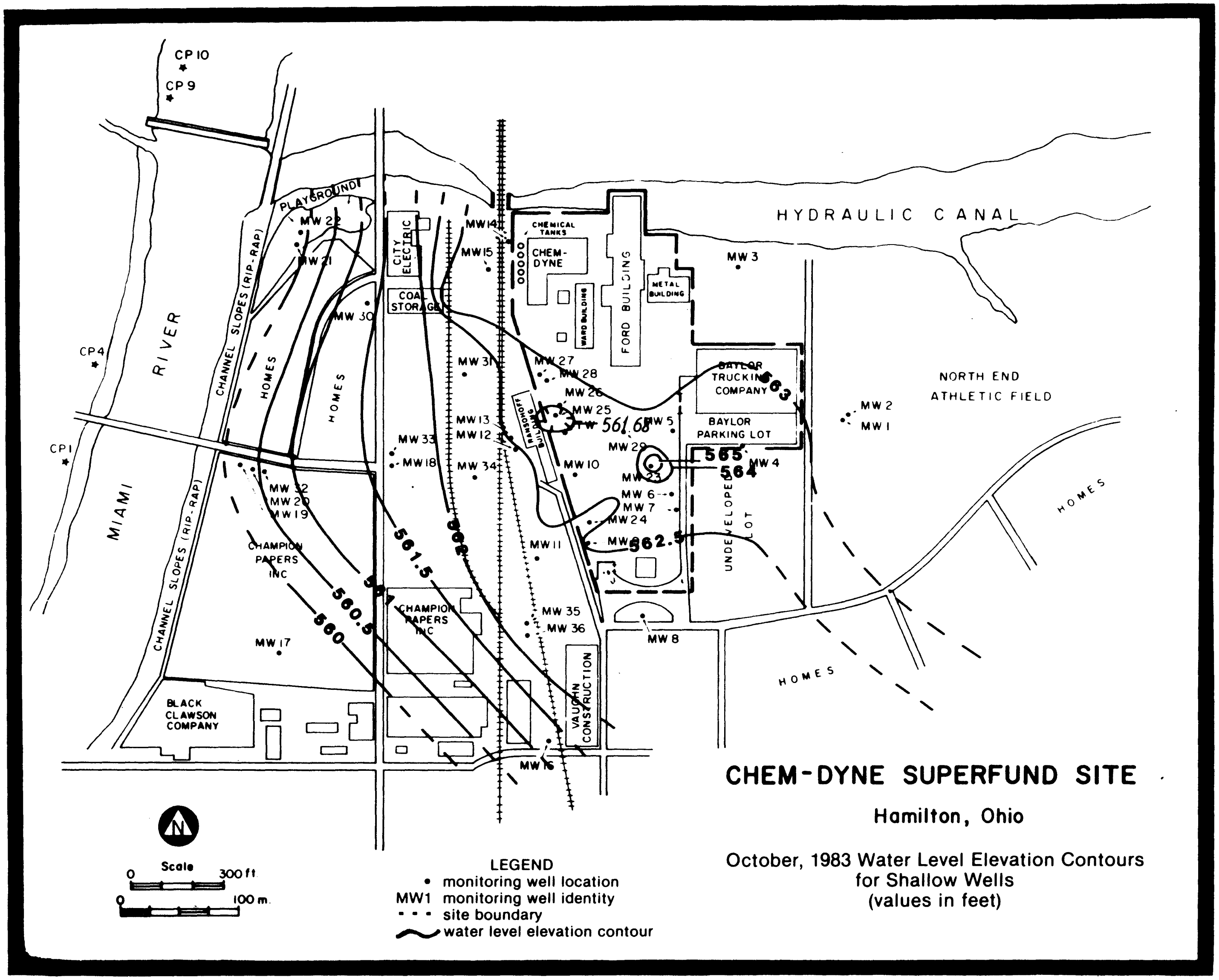


Figure 4.30. October, 1983 Water Level Elevation Contours at Chem-Dyne Site Using Shallow Well Data Only in Feet MSL

had to be drilled onsite for reasons of potential liability and lack of property access elsewhere. The drillers were substantially slowed in their work by the need to don air-tanks when particularly contaminated subsoils were encountered; the emission of volatile fumes from the borehole presented unacceptable health risks. Since the waters that would be pumped were expected to be contaminated, it was necessary to construct ten large temporary holding tanks (100,000 gallons each) onsite to impound the waters for testing and possible treatment prior to being discharged to the local sewer system (CH_2M-Hill, 1984a).

The costs and difficulty of preparing for and conducting the test were worth the effort, however. The water levels in thirty-six monitoring wells were observed during the test and yielded a very detailed picture of transmissivity variations (Figures 4.31 and 4.32), which has been used to help explain the unusual configuration of the plume such as shown in Figures 4.33, 4.34, and 4.36 (Figure 4.35 is an updated location map for 1985 data presented in Figure 4.36 and some later figures); and which was used to guide the design of a pump-and-treat system. Storage coefficients were also estimated; and though the short duration of the test (14 hours) did not allow for many definitive estimates to be obtained, it was clear that qualitative confirmation of the generally non-artesian (water-table) nature of the aquifer beneath the site was confirmed, as were the increasingly artesian conditions from the west edge of the site towards the river.

An automated data acquisition system (In-Situ, Inc.'s SE-200A™ computer-controlled pressure-transducer system) was used to monitor the water levels and provide real-time drawdown plots of 19 of the 36 wells

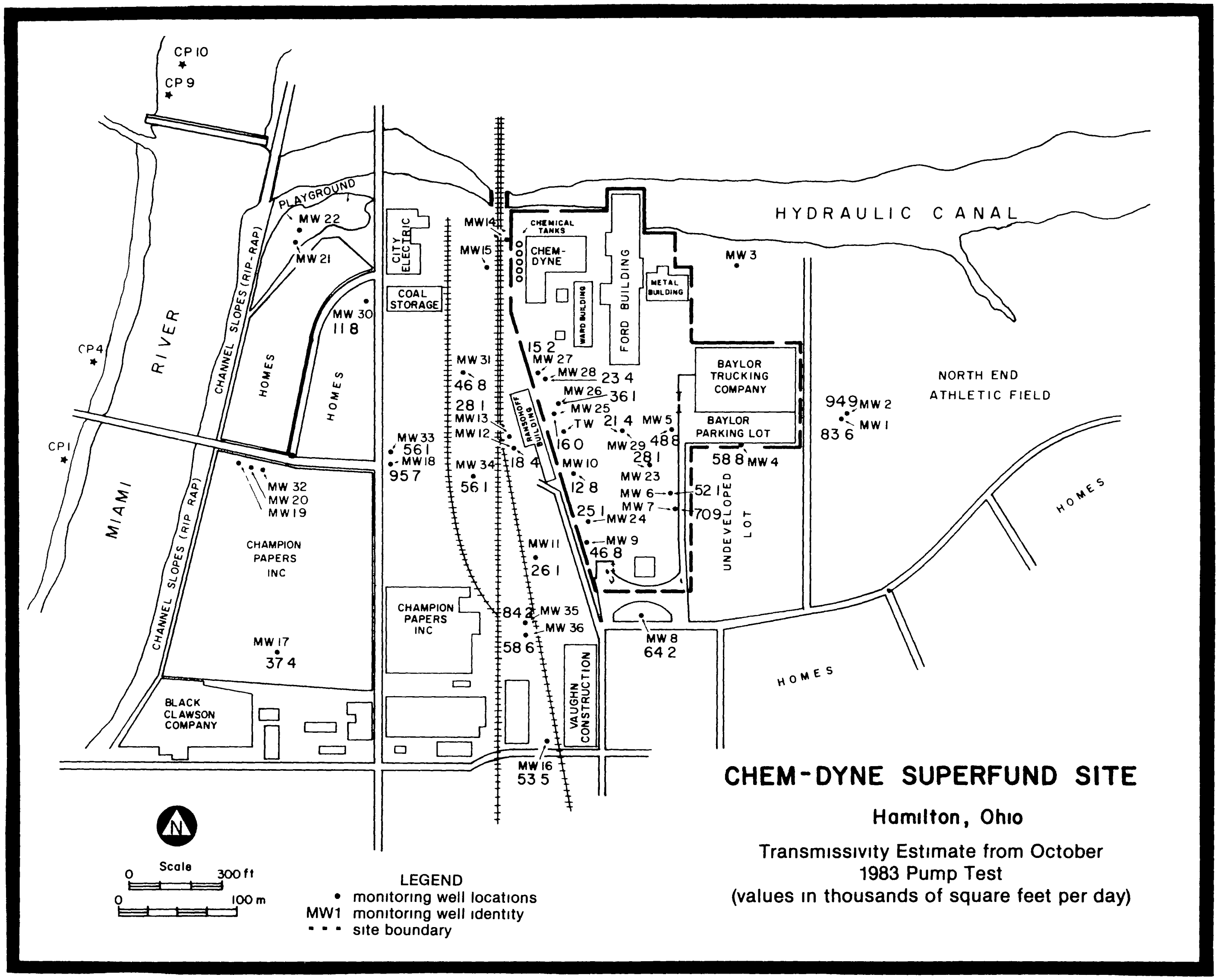


Figure 4.31. Transmissivity Estimates Obtained from October,1983 Pump Test at Chem-Dyne Site, in Thousands of Square Feet/Day (Plotted Values of Estimates from All Wells)

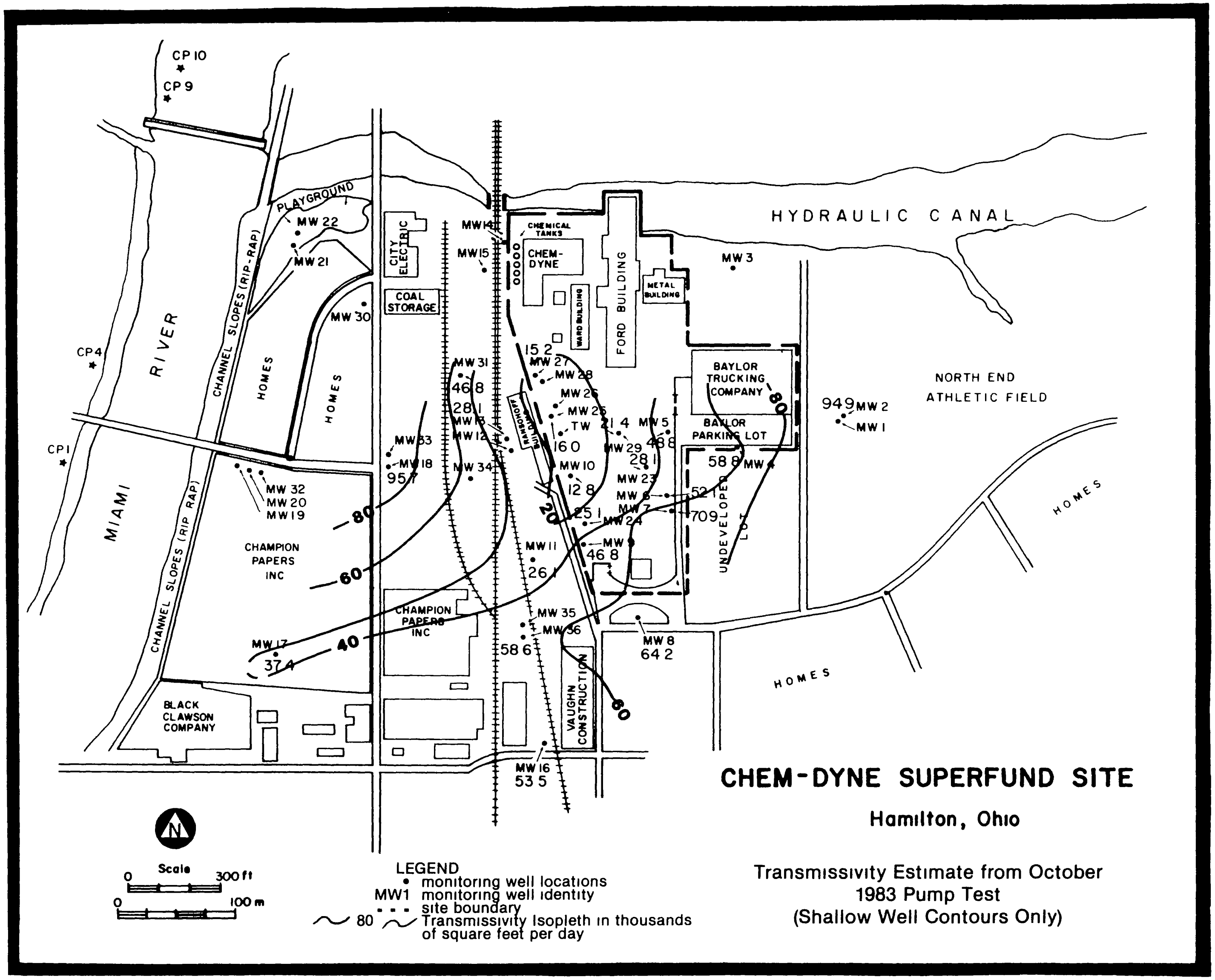


Figure 4.32. Transmissivity Estimates Obtained from October, 1983 Pump Test at Chem-Dyne Site, in Thousands of Square Feet/Day (Contours of Values Obtained from Shallow Wells Only)

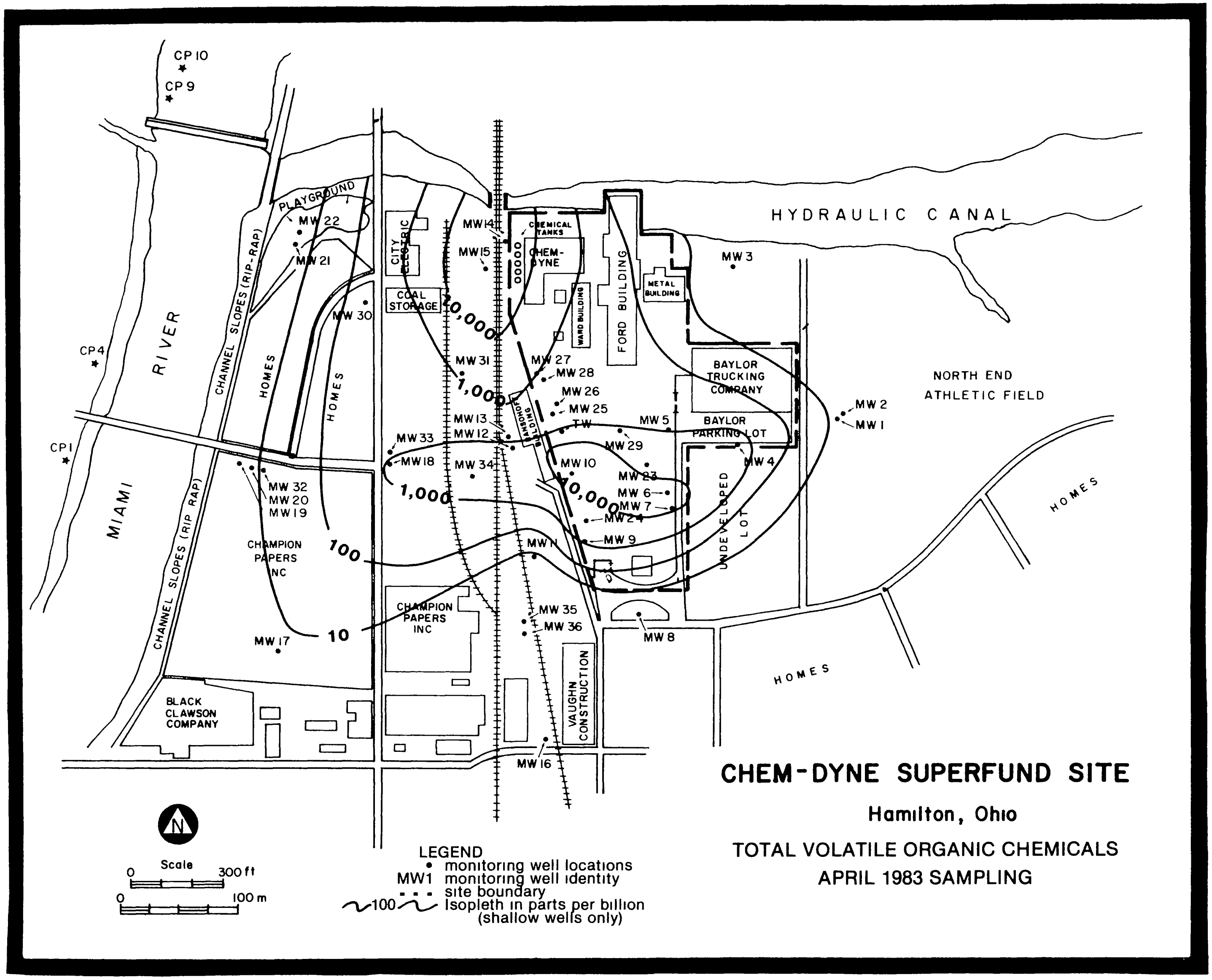


Figure 4.33. April, 1983 Total VOC Concentration Contours (ppb) at Chem-Dyne Site, Using Shallow Well Data Only

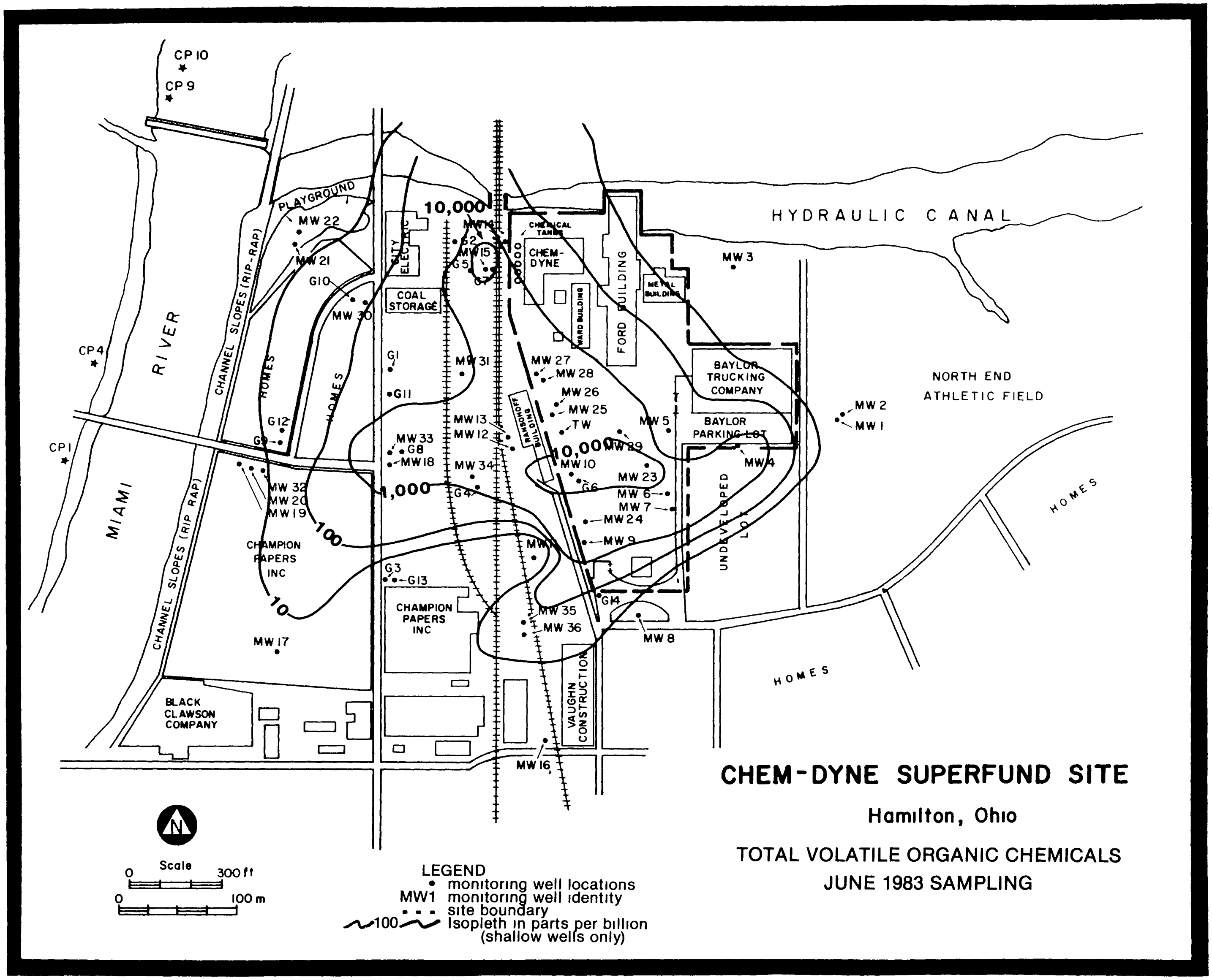


Figure 4.34. June, 1983 Total VOC Concentration Contours (ppb) at Chem-Dyne Site, Using Shallow Well Data Only

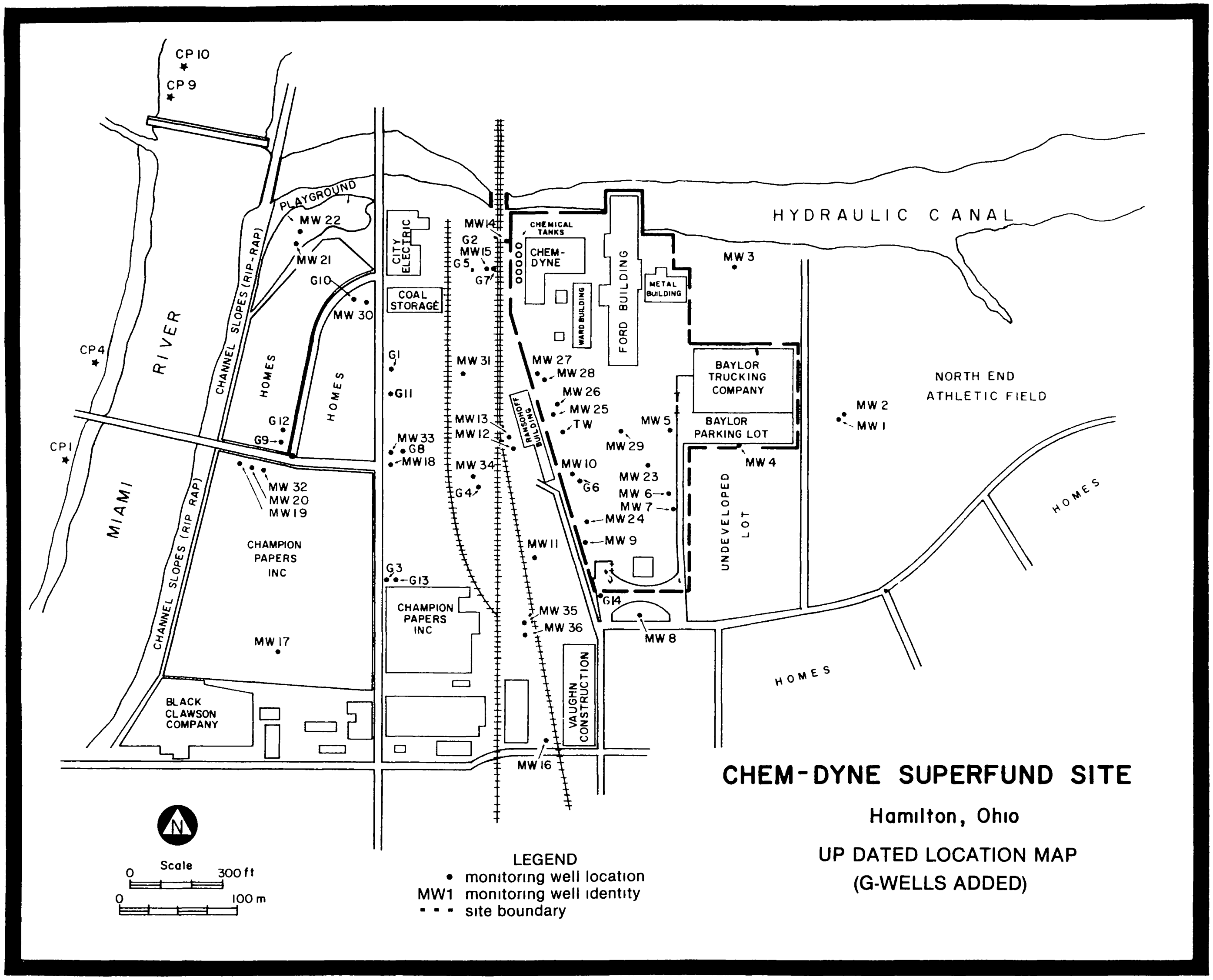


Figure 4.35. Updated Location Map for Case History No.4

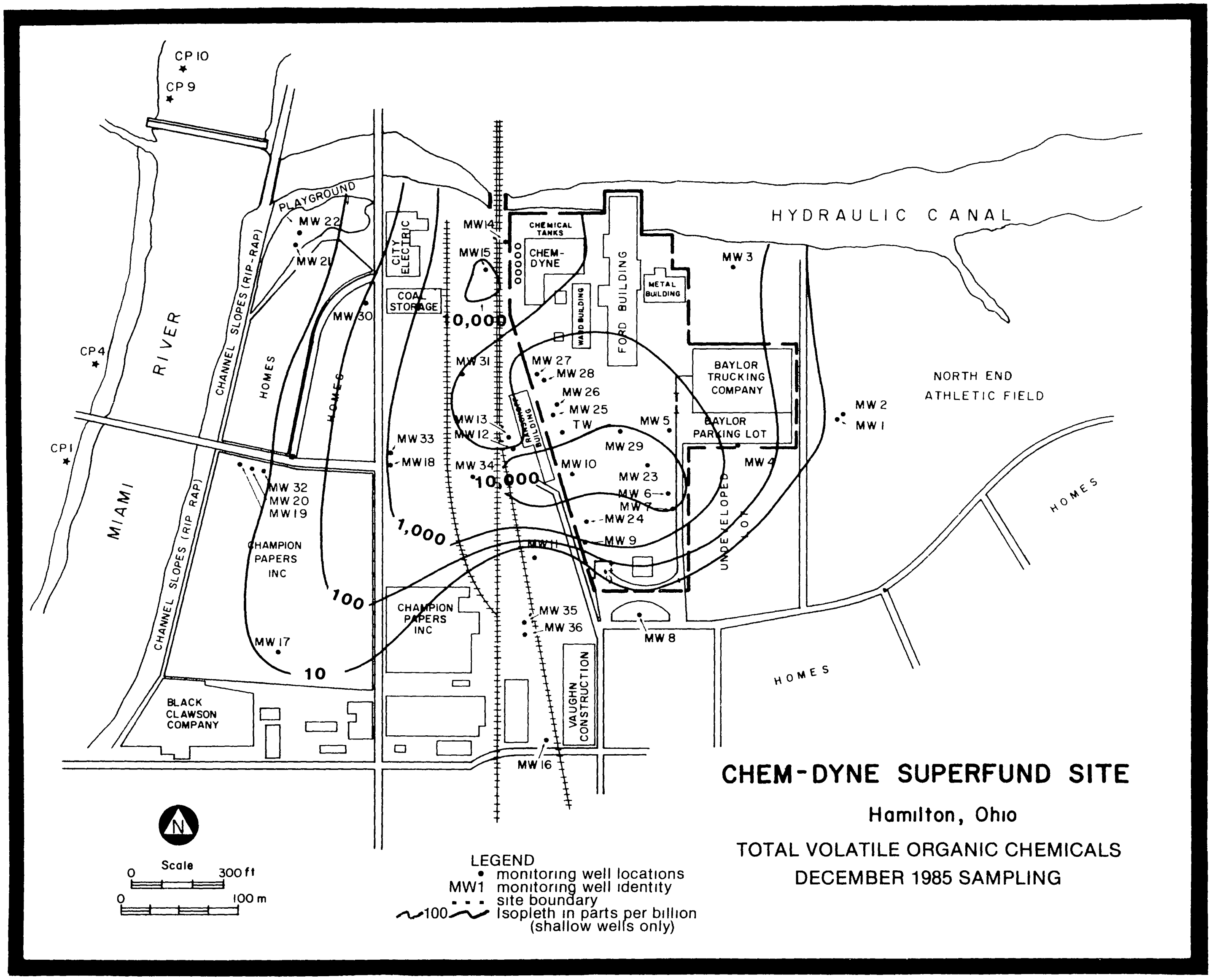


Figure 4.36. December, 1985 Total VOC Concentration Contours (ppb) at Chem-Dyne Site, Using Shallow Well Data Only

(Table 4.3), greatly enhancing the information obtained with only minimal manpower requirements. Typical outputs from the system are shown in Figure 4.37, where it can be seen that the rapidity of the measurements obtained is as much an advantage as is the lowered manpower requirements. The benefits from conducting the pump test cannot be overemphasized; qualitative confirmation of lithologic information and semi-quantitative estimation of crucial parameters were obtained. For example, low transmissivity values were found to occur on a north-south axis at the west fenceline of the site (Figure 4.32), where significant findings of clays were documented during drilling (Figure 4.22).

4.5.6 Anisotropic Flow Biases

Onsite transmissivity estimates from a trio of wells (MW-23, MW-26, and MW-29) indicated a 2:1 anisotropic bias toward the river as opposed to downvalley, whereas nearer the river a second trio of wells (MW-28, MW-33, and MW-35) yielded estimates for which the bias appeared to be 10:1 downvalley (CH_2M-Hill, 1984a). These trends coincide with the nature of the system; there are few clays onsite and east of the site, from whence recharge waters flow toward the river, so that one expects the indicated riverward preference there. There are significant clays at the west fenceline of the site and adjacent to the river that have lenticular shapes paralleling the river, so that one expects a strong downvalley bias there. These indications mean that flow would not proceed at an average velocity perpendicular to the water level contours, which would be west from the northern half of the site, and south to southwest from the southern half of the site (Figure 4.24). Instead, flow would move

Table 4.3. Instruments Used to Measure Water Levels During the October, 1983 Pump Test at the Chem-Dyne Site

Well No.	Radial Dist. (feet)	Water Level (feet MSL)	Method of Measurement (type, field unit)
MW1	957	563.68	manual, electric probe
MW2	965	563.74	manual, electric probe
MW3	848	563.96	automatic, float-type
MW4	537	563.27	manual, electric probe
MW5	313	563.31	manual, electric probe
MW6	420	564.40	manual, electric probe
MW7	480	563.30	manual, electric probe
MW8	740	563.01	manual, electric probe
MW9	487	563.08	manual, electric probe
MW10	186	563.29	auto., pressure transducer
MW11	502	562.90	auto., pressure transducer
MW12	232	562.39	auto., pressure transducer
MW13	232	563.19	auto., pressure transducer
MW14	701	-	dry - no data collected
MW15	611	563.10	auto., pressure transducer
MW16	1275	562.47	manual, electric probe
MW17	1518	560.03	manual, electric probe
MW18	692	562.67	manual, electric probe
MW19	1204	559.80	automatic, float-type
MW20	1225	562.10	automatic, float-type
MW21	1259	561.29	manual, electric probe
MW22	1261	559.95	auto., pressure transducer
MW23	298	563.07	auto., pressure transducer
MW24	398	563.07	auto., pressure transducer
MW25	53	563.04	auto., pressure transducer
MW26	62	562.96	auto., pressure transducer
MW27	272	563.13	auto., pressure transducer
MW28	248	562.99	auto., pressure transducer
MW29	167	563.23	auto., pressure transducer
MW30	993	561.25	manual, electric probe
MW31	465	562.78	automatic, float-type
MW32	1236	559.56	auto., pressure transducer
MW33	690	562.06	auto., pressure transducer
MW34	454	562.29	auto., pressure transducer
MW35	651	562.93	auto., pressure transducer
MW36	696	562.69	manual, electric probe
Pump Well	(ref.pt.)	562.97	auto., pressure transducer

```
    Tacoma Well 12A Pump Test         ---------DRAWDOWN REPORT---------
              Run 1
            05/29/86                  Started at 1100
                                      Lasted 1208.6 min
           SE200A DATA
       constant rate test


        TRANSDUCER TABLE

Input 1: CBW-06                       Input 1 (feet):
   Transducer s/n: 713
   Scale factor:    10.08             Time   ET (min)    level   Δlevel
   Initial level:   222.54 feet       ----  ---------  -------  -------
                                      1100      0.000   222.54     0.0
                                      1100      0.257   227.70    -5.1
Input 2: CBW-06                       1100      0.340   228.26    -5.7
   Transducer s/n: 265                1100      0.424   228.67    -6.1
   Scale factor:    50.29             1100      0.507   229.01    -6.4
   Initial level:   222.54 feet       1100      0.590   229.24    -6.7
                                      1100      0.674   229.28    -6.7
                                      1100      0.757   229.14    -6.6
Input 3: CBW-08                       1100      0.840   228.99    -6.4
   Transducer s/n: 720                1101      0.924   228.89    -6.3
   Scale factor:    10.08             1101      1.007   228.82    -6.2
   Initial level:   222.13 feet       1101      1.394   228.74    -6.2
                                      1101      1.727   228.75    -6.2
                                      1102      2.060   228.77    -6.2
Input 4: CBW-01                       1102      2.394   228.80    -6.2
   Transducer s/n: 717                1102      2.727   228.84    -6.3
   Scale factor:    10.1              1103      3.060   228.87    -6.3
   Initial level:   220.89 feet       1103      3.394   228.89    -6.3
                                      1103      3.727   228.90    -6.3
                                      1104      4.060   228.92    -6.3
Input 5: CBW-05                       1104      4.394   228.94    -6.4
   Transducer s/n: 261                1104      4.727   228.95    -6.4
   Scale factor:    50.24             1105      5.060   228.96    -6.4
   Initial level:   221.75 feet       1105      5.394   228.97    -6.4
                                      1105      5.727   228.99    -6.4
                                      1106      6.060   229.00    -6.4
Input 6: CBW-05                       1106      6.394   229.01    -6.4
   Transducer s/n: 712                1106      6.727   229.03    -6.4
   Scale factor:    10.05             1107      7.060   229.03    -6.4
   Initial level:   221.75 feet       1107      7.394   229.04    -6.5
                                      1107      7.727   229.05    -6.5
   FAST DATA                          1108      8.060   229.06    -6.5
                                      1108      8.394   229.06    -6.5
                                      1108      8.727   229.07    -6.5
          PUMP SCHEDULE               1109      9.060   229.08    -6.5
                                      1109      9.394   229.09    -6.5
Drawdown for 1440 min                 1109      9.727   229.10    -6.5
Pump at 3500GPM                       1110     10.060   229.10    -6.5
Pump set at 130 feet                  1112     12.204   229.14    -6.6
                                      1114     14.204   229.16    -6.6
                                      1116     16.173   229.18    -6.6
        SAMPLING SCHEDULE             1118     18.173   229.20    -6.6
                                      1120     20.178   229.21    -6.6
0-10          sec  @    1 sec         1122     22.173   229.24    -6.7
10-60         sec  @    5 sec         1124     24.328   229.24    -6.7
1-10          min  @   20 sec         1126     26.173   229.25    -6.7
10-100        min  @    2 min         1128     28.243   229.26    -6.7
100-1000      min  @   20 min         1130     30.185   229.28    -6.7
1000-10000    min  @   60 min         1132     32.185   229.28    -6.
10000-99999   min  @  200 min         1134     34.185   229.30    -6.7
```

Figure 4.37. Typical Output from SE-200A Unit During a Pump Test

westerly first and then southerly as it neared the river. By the time areas adjacent to the river had been reached the net position would be roughly the same, but the path taken to get there would be strikingly different; the contaminant masses leaving both the northern and southern halves of the site would be concentrated in west trending tongues. Moreover, the future travel paths would not be the same with the anisotropic bias interpretation as with a presumption of isotropicity.

The distributions of contaminants observed at Chem-Dyne seem to support this anisotropic bias interpretation. The highest concentrations lie along an axis that does not appear to be influenced by the southerly components of flow expressed by the water-level contours offsite (Figures 4.32 to 4.34). While it is true that the pumpage of major production wells on the other side of the river offer a nominal explanation for this in the abstract, the actual water level contours contradict the notion that the industrial wells dominate the entire flow field. Again, this goes back to the discussions of capture zones; the industrial wells will indeed affect all of the ground-water flowlines locally, but they may not capture all of them. It is likely that the plume is bifurcating near the river, with one member travelling on flowlines captured by the industrial wells, and a second member tangentially affected, but eventually released to continue on down the valley with the rest of the aquifer waters.

4.5.7 Field Evidence for Biotransformations

Finally, the distribution patterns of contaminant species that emerged from the investigations at Chem-Dyne were made understandable by considering research results and theories regarding chemical and microbiological influences. Contaminant distribution maps from samples taken at the end of the field investigation (October, 1983), only months after the last drums of solvents had been removed from the site suggested that the transformation of tetrachloroethene (Figure 4.38) to less halogenated daughter products, such as trichloroethene (Figure 4.39), dichloroethene (Figure 4.40), and vinyl chloride / monochloroethene (Figure 4.41) was taking place.

In such circumstances, one would expect to see the progressive disappearence of tetrachloroethene and successive increases in the concentrations and extent of potential daughter products. This seems to be the case at Chem-Dyne, by examination of the October, 1983 data. One might argue that too little vinyl chloride is observed (Figure 4.41) to show the full series of degradation expected, but there are plausible reasons why the distributions might be as shown. For example, with a continuous source input the concentrations of tetrachloroethene might be high enough that there would be no need for further biotransformation of daughter products, because an ample food supply is available in the parent material. Alternatively, the concentrations of tetrachloroethene with continuous source inputs might indeed be so high as to limit biotransformation by toxic effects. Since much remains to be done in terms of defining the relative kinetics of the various transformations in this sequence, it is impossible to make rigorous conclusions with regard to these possibilities.

But consider the data obtained during a chemical sampling conducted two years later, in preparation for activation of the pump-and-treat

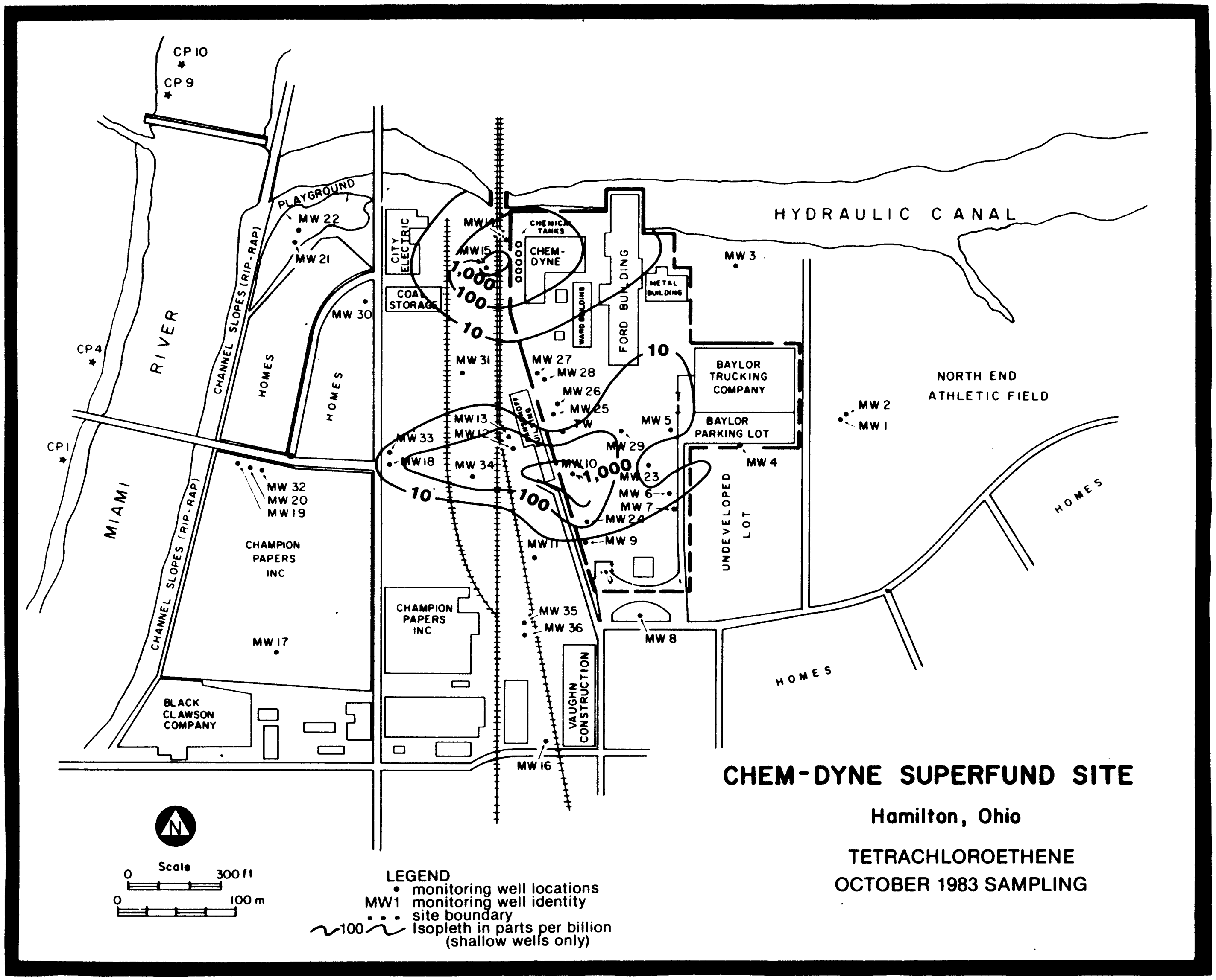


Figure 4.38. October, 1983 Tetrachloroethene Concentration Contours (ppb) at Chem-Dyne Site, Using Shallow Well Data Only

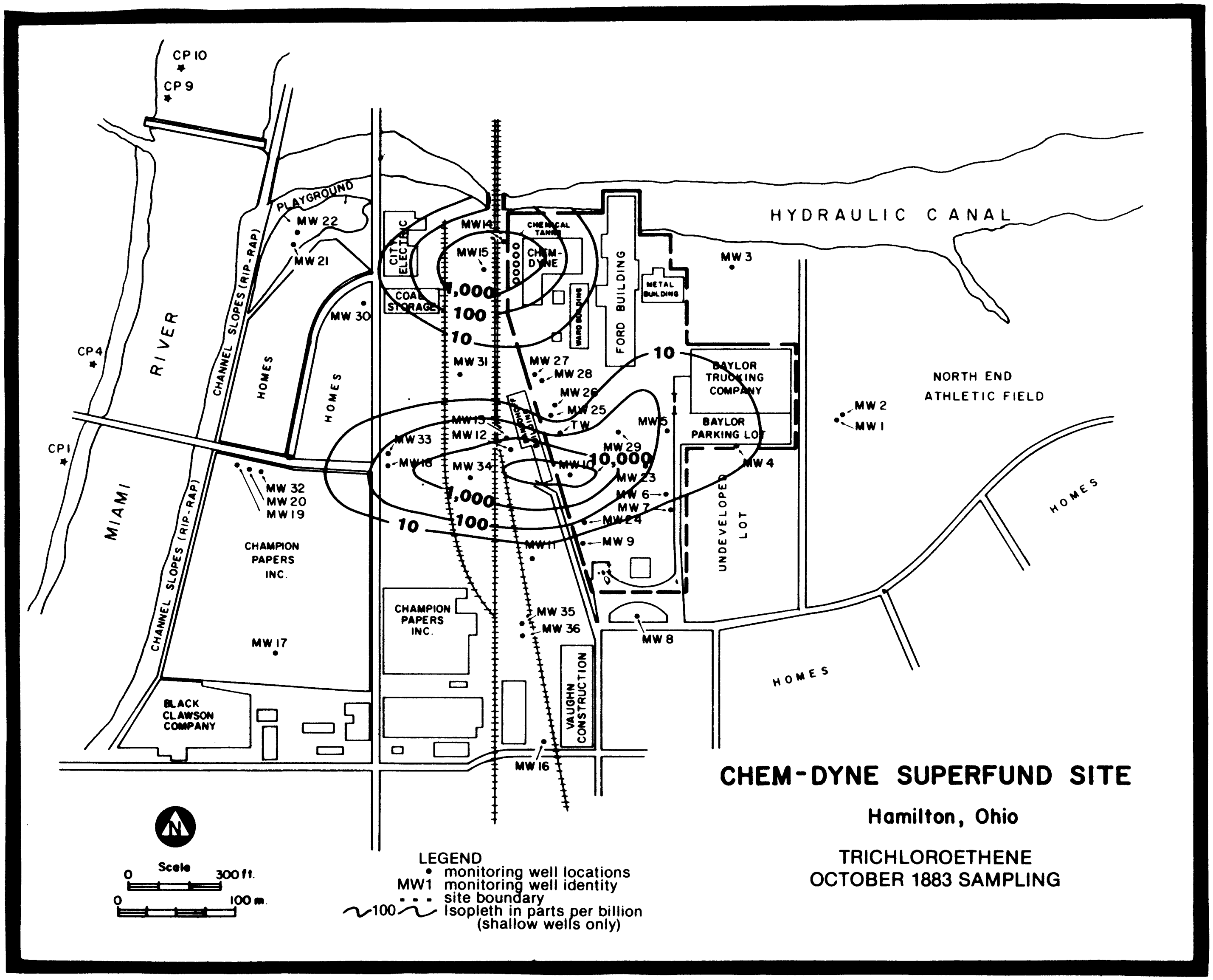


Figure 4.39. October, 1983 Trichloroethene Concentration Contours (ppb) at Chem-Dyne Site, Using Shallow Well Data Only

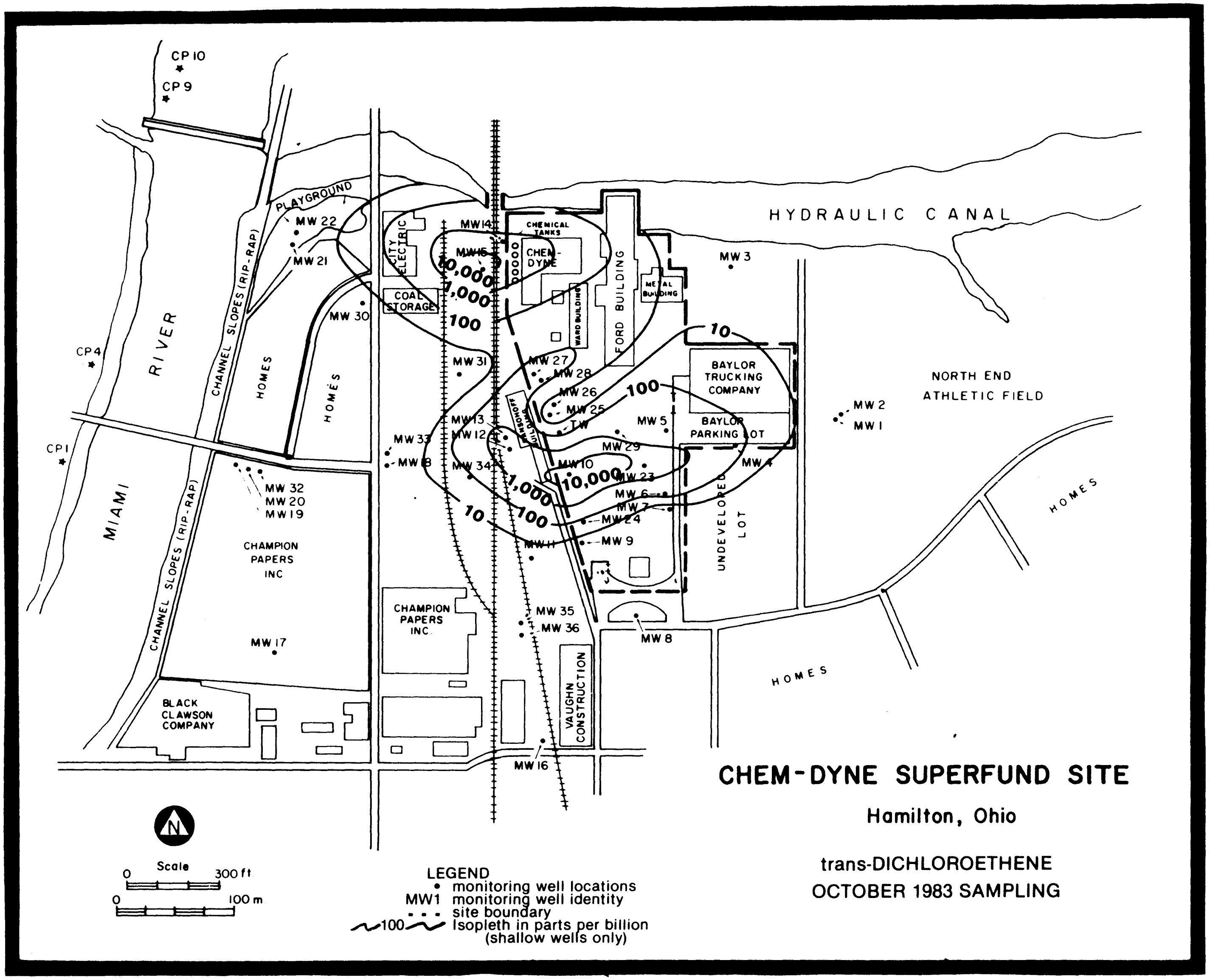


Figure 4.40. October, 1983 trans-Dichloroethene Concentration Contours (ppb) at Chem-Dyne Site, Using Shallow Well Data Only

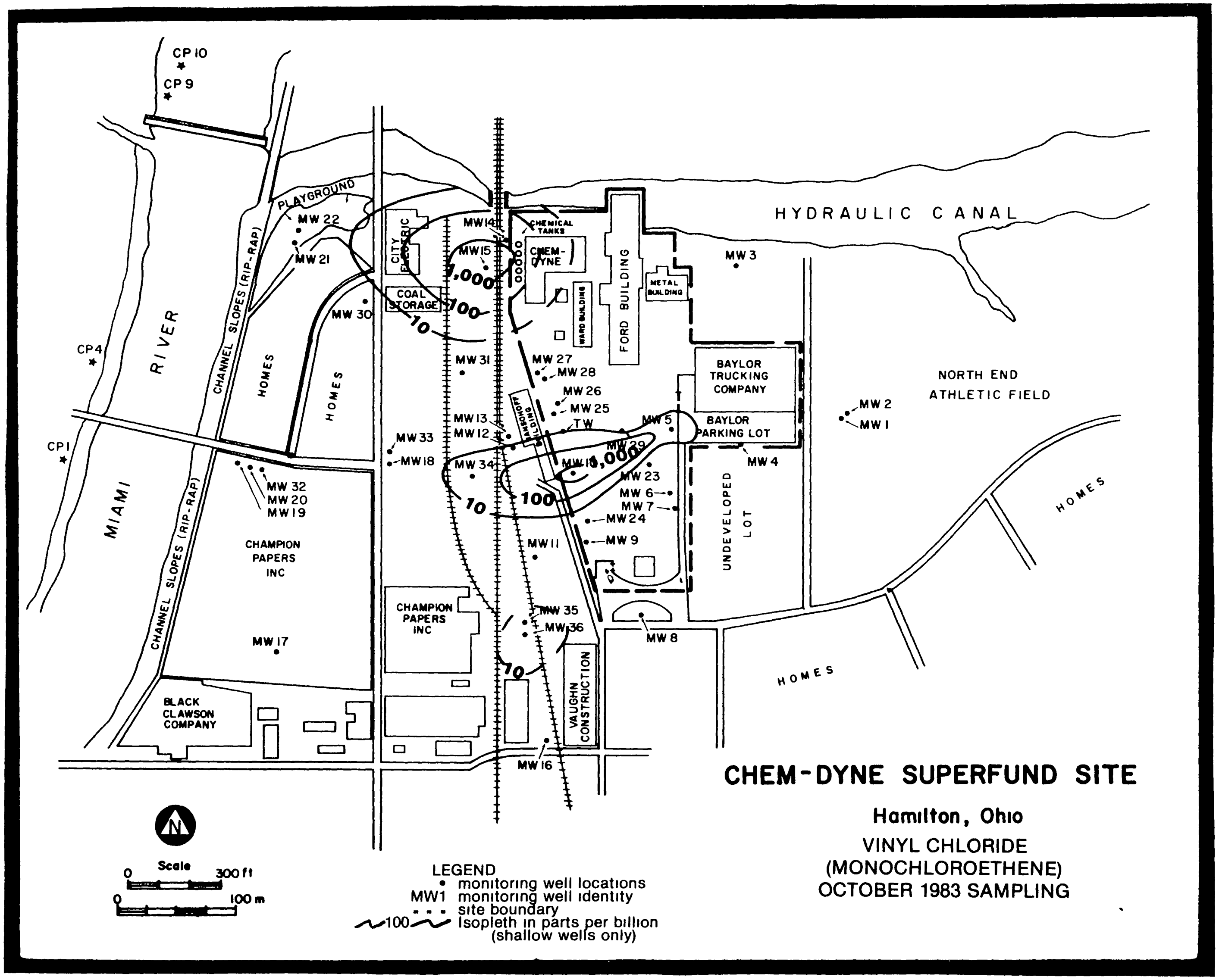


Figure 4.41. October, 1983 Vinyl Chloride Concentration Contours (ppb) at Chem-Dyne Site, Using Shallow Well Data Only

system that will be used to remediate the plume (Figures 4.42 to 4.45). It has been at least two years of freedom from surface inputs of solvents; and two years of healthy rainwater flushing the unsaturated zone of stored residues. What is now apparent is that the daughter products (Figures 4.43 to 4.45) contain much greater mass than is contained in the tetrachloroethene contours (Figure 4.42), and they are spread over significantly greater areas. The increase in the vinyl chloride component of the plume is staggering (compare Figure 4.45 to Figure 4.41). These data are highly suggestive of active degradation of tetrachloroethene to its possible daughter products. Knowledge of this kind of possible transformation should be valuable to those attempting to cost and design treatment systems, since treatment efficiencies vary with the contaminant and its contribution to the overall loading. Vinyl chloride is much more volatile and much easier to remove than tetrachloroethene.

The relative rates of movement of other common solvents, like benzene (Figures 4.46 and 4.47) and chloroform (Figures 4.48 and 4.49), generally conformed to predictions based on sorption principles. Again this is useful in a practical sense. The remediation efforts made use of these contaminant transport theories in estimating the capacity of the treatment system needed and the length of time necessary to remove residuals from the aquifer solids (CH_2M-Hill, 1984b).

4.5.8 The Role of Mathematical Models

During the latter stages of negotiations with the Potentially Responsible Parties (PRP's), Ohio State government contractors prepared

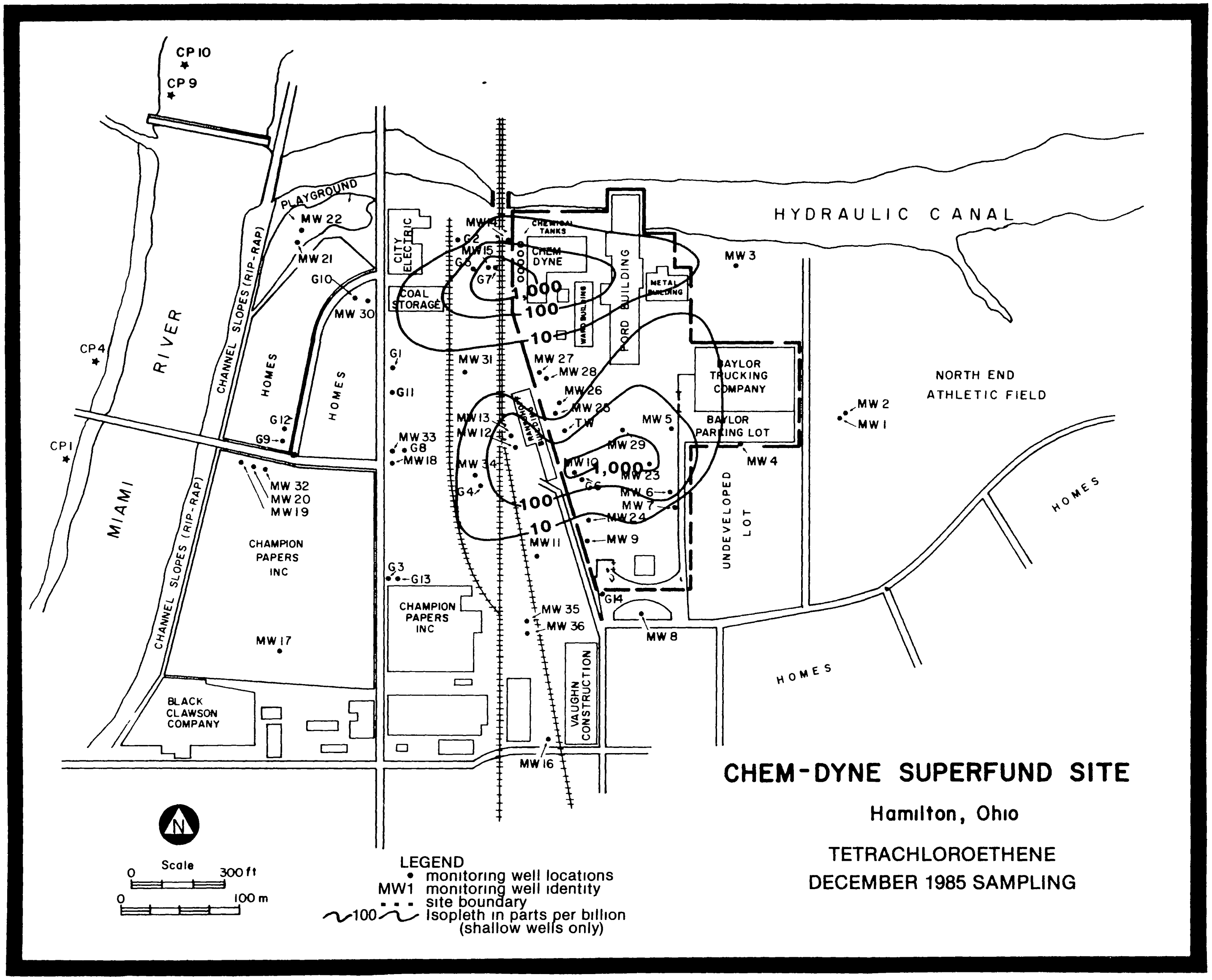


Figure 4.42. December, 1985 Tetrachloroethene Concentration Contours (ppb) at Chem-Dyne Site, Using Shallow Well Data Only

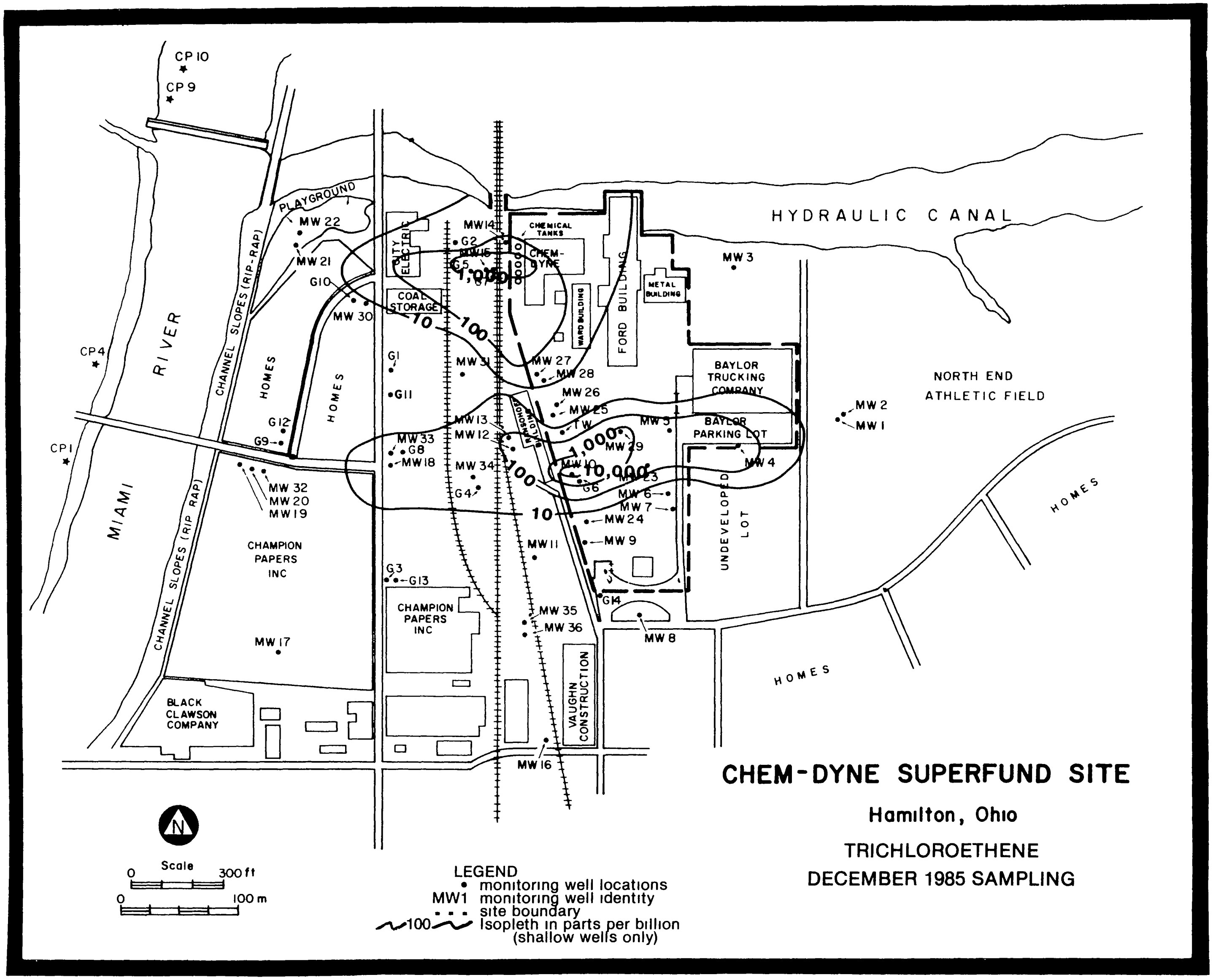


Figure 4.43. December, 1985 Trichloroethene Concentration Contours (ppb) at Chem-Dyne Site, Using Shallow Well Data Only

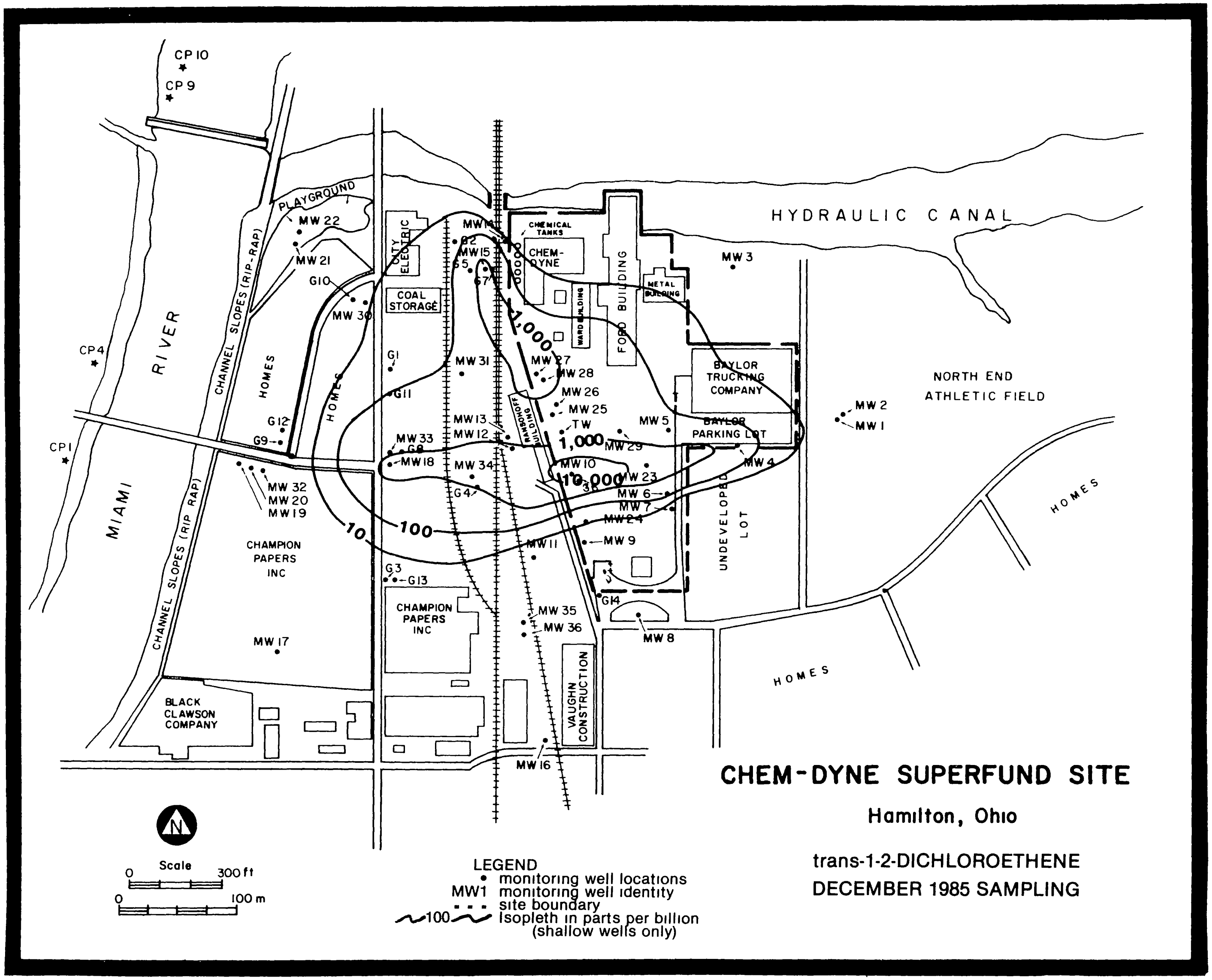


Figure 4.44. December, 1985 trans-Dichloroethene Concentration Contours (ppb) at Chem-Dyne Site, Using Shallow Well Data Only

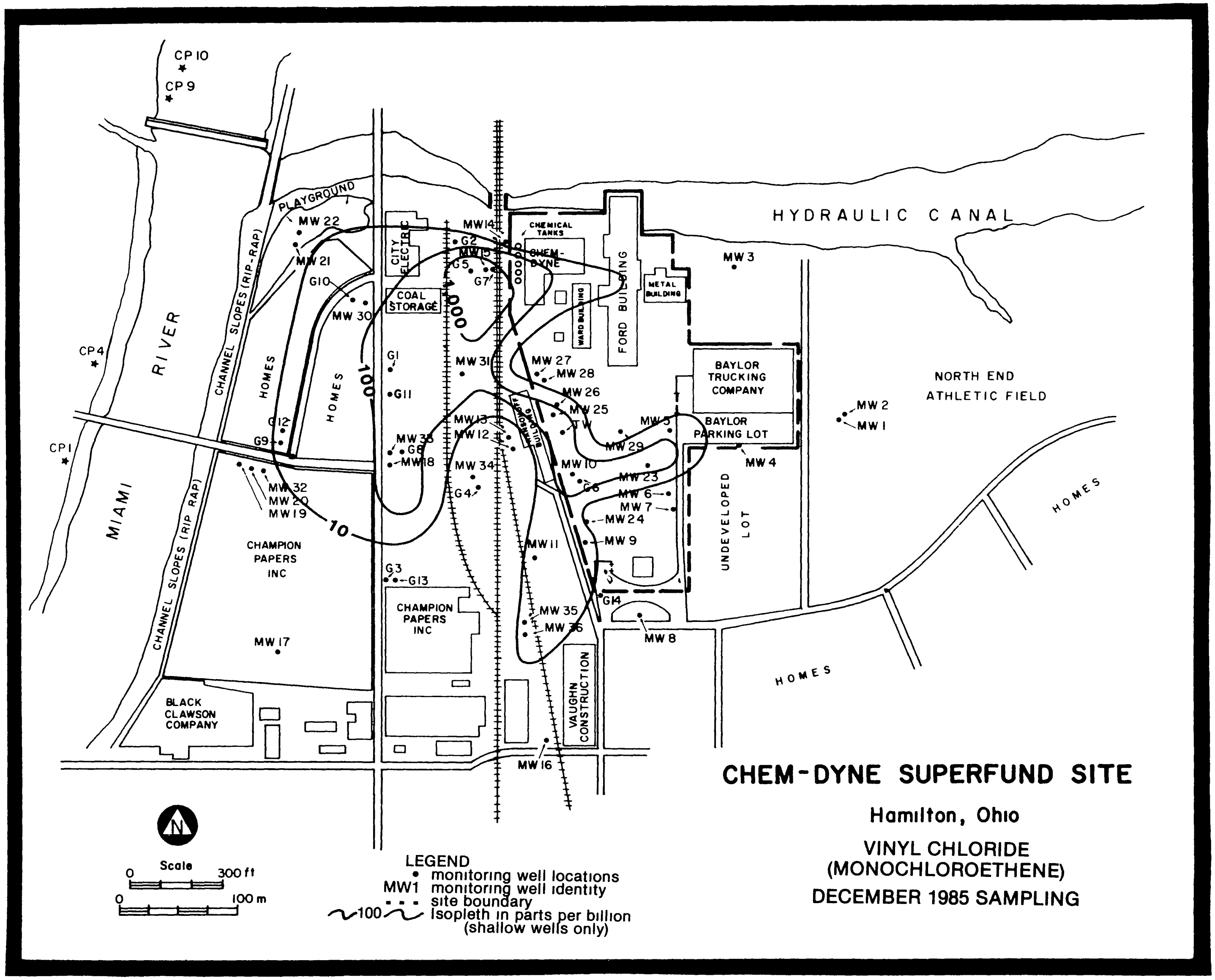


Figure 4.45. December, 1985 Vinyl Chloride Concentration Contours (ppb) at Chem-Dyne Site, Using Shallow Well Data Only

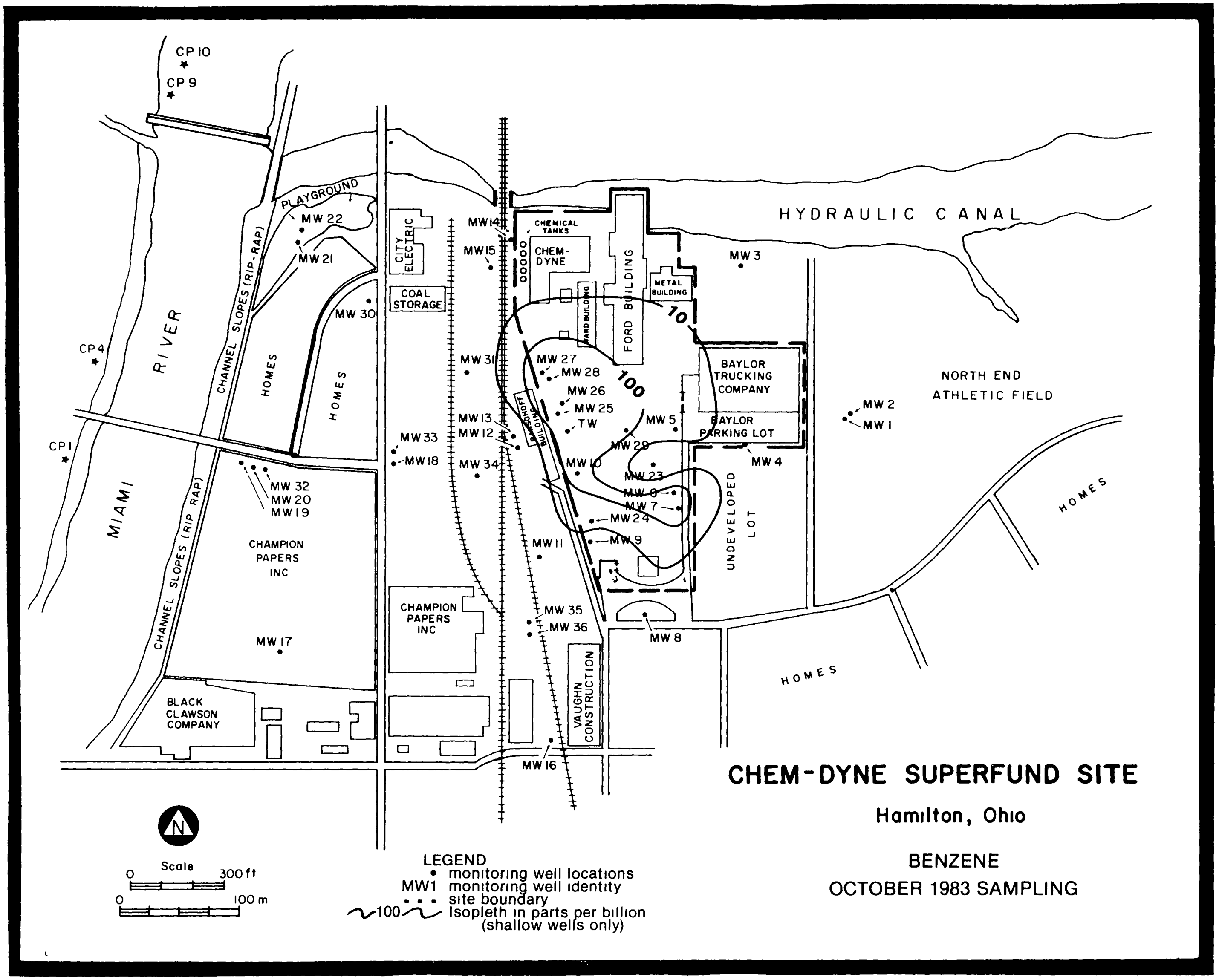


Figure 4.46. October, 1983 Benzene Concentration Contours (ppb) at Chem-Dyne Site, Using Shallow Well Data Only

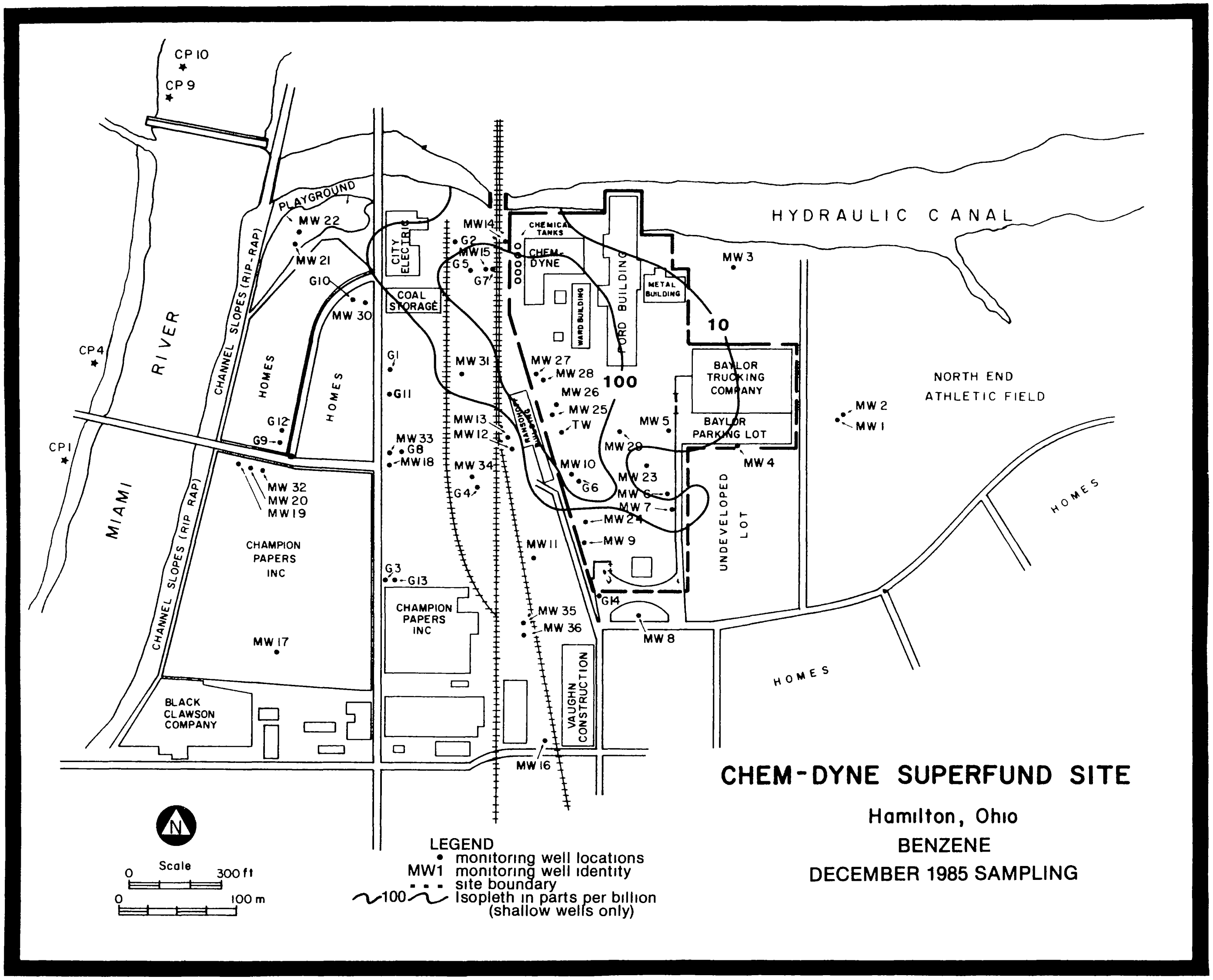


Figure 4.47. December, 1985 Benzene Concentration Contours (ppb) at Chem-Dyne Site, Using Shallow Well Data Only

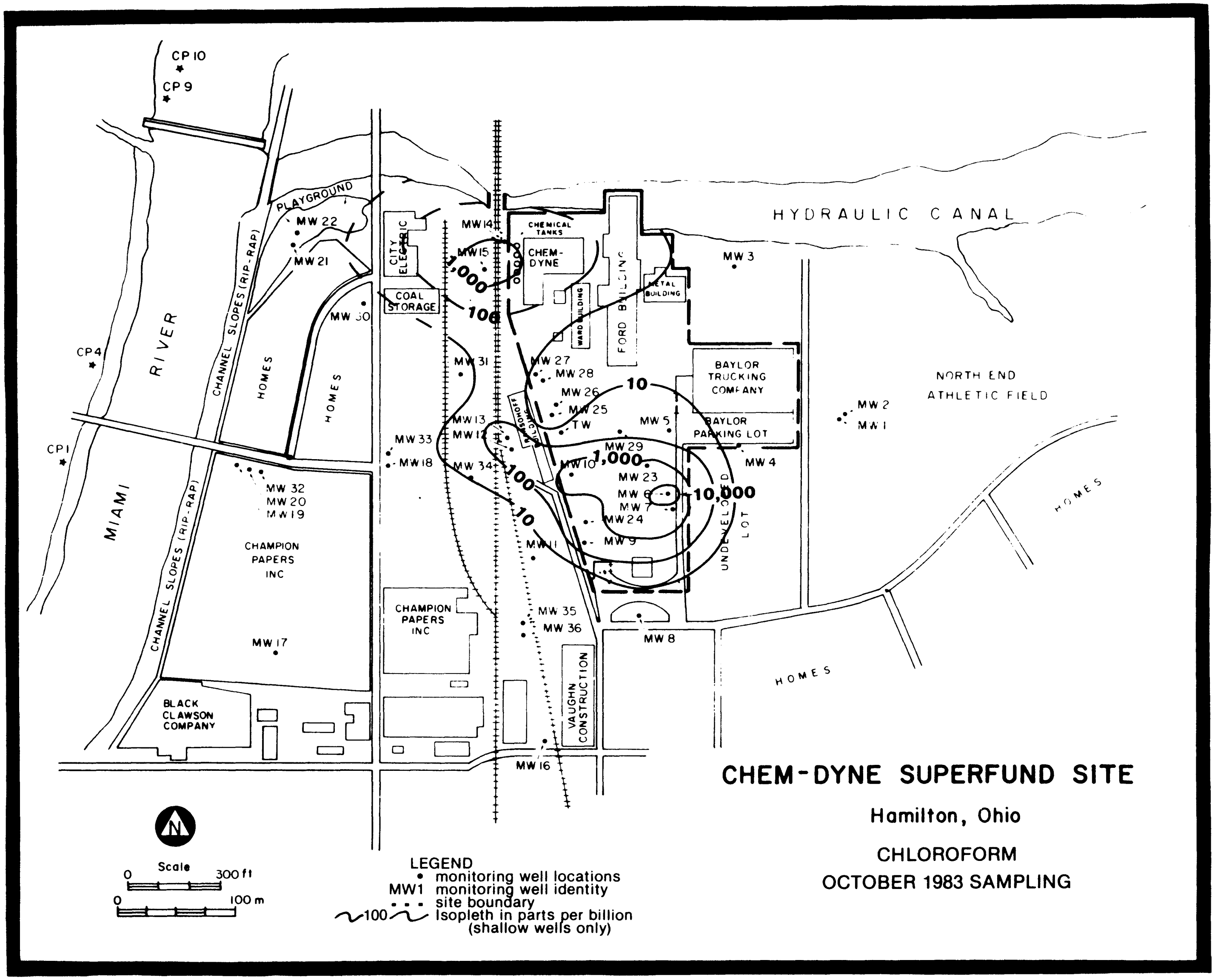


Figure 4.48. October, 1983 Chloroform Concentration Contours (ppb) at Chem-Dyne Site, Using Shallow Well Data Only

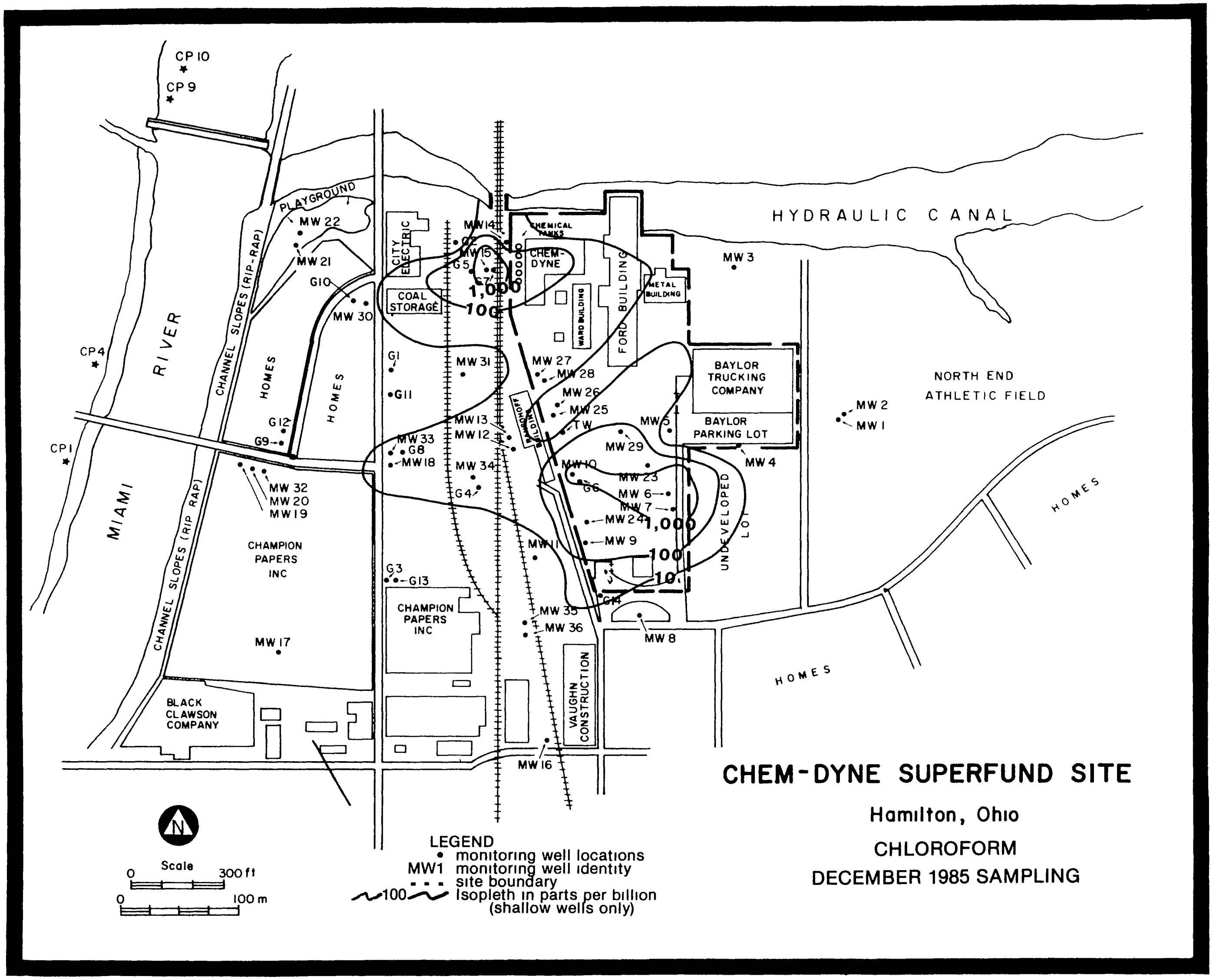


Figure 4.49. December, 1985 Chloroform Concentration Contours (ppb) at Chem-Dyne Site, Using Shallow Well Data Only

mathematical models of the flow system and contaminant transport at Chem-Dyne (GeoTrans, 1984). These were used to estimate the possible direction and rate of migration of the plume in the absence of remediation, the mass of contaminants removed during various remedial options, and the effects of sorption and dispersion on those estimates. Because of the wide range of sorption properties associated with the variety of contaminants found in significant concentrations it was necessary to select values of retardation constants that represented the likely upper- and lower-limits of sorptive effects. It was also necessary to estimate or assume the values of other parameters known to affect transport processes, such as dispersion coefficients.

While the developers of the models would be the first to acknowledge the large uncertainties associated with those modeling efforts due to lack of information about the actual history of chemical inputs and other important data, there was agreement between the government and PRP technical experts that the modeling efforts had been very helpful in assessing the magnitude of the problem and in determining minimal requirements for remediation. Consequently, modeling efforts will continue at Chem-Dyne. Data generated during the remediation phase will be used to refine models in an ongoing process so that the effectiveness of the remedial action can be evaluated properly.

CHAPTER V

GROUND-WATER CONTAMINATION ASSESSMENTS IN PERSPECTIVE

5.1 Perspectives for Practical Applications

The evolution of concepts pertinent to investigating and predicting the movement of contaminants in the subsurface has been described. An appreciation for the fact that these concepts are evolving is important because there seems to be widespread overconfidence by decisionmakers regarding the ability to predict transport and fate of contaminants in the subsurface. The preceeding discussion should place in perspective the advances that have been made in recent years, by illustrating the practical implications of conceptual uncertainties that remain unresolved.

5.1.1 Site Investigation Approaches

From a practical perspective, it appears that there is too little emphasis in most ground-water contamination assessments on obtaining detailed information about the potential pathways and mechanisms that affect transport. Additional effort devoted to site-specific characterizations of natural process parameters, rather than relying almost exclusively on chemical analyses of ground-water samples, can significantly improve the quality and cost-effectiveness of the remedial actions at hazardous waste sites. To underscore this point, condensed summaries are provided of the principal activities, benefits, and shortcomings of three possible site characterization approaches: conventional (Table 5.1), state of the art (Table 5.2), and state of the science (Table 5.3).

To further illustrate this, a qualitative assessment of desired trade-offs between characterization and clean-up costs is presented in Figure 5.1. As illustrated there, some investments in specialized equipment and personnel will be necessary to make the transition to more sophisticated approaches, but those investments will be more than paid back in reduced clean-up costs. The maximum return on increased investments is expected for the state-of-the-art approach, and will diminish as the state-of-the-science approach is reached because highly specialized equipment and personnel are not widely available. It is vitally important that this philosophy be considered, because the probable benefits in lowered total costs, health risks, and time can be substantial.

5.1.2 A Hypothetical Example

It is helpful to examine possible scenarios that might result from the different site investigation approaches just outlined. Figure 5.2 depicts a situation fashioned somewhat after the Chem-Dyne site discussed in the preceeding chapter. A number of features have been added to facilitate this discussion, however, so that the following is not relevant to Chem-Dyne specifically. For example, three major plumes are shown in Figure 5.2, an acids plume (e.g., from electrolytic plating operations), a phenols plume (e.g., from a creosoting operation that used

Table 5.1. Conventional Approach to Site Characterization

SITE CHARACTERIZATION
CONVENTIONAL APPROACH

ACTIONS TYPICALLY TAKEN

Install a few dozen shallow monitoring wells
Sample and analyze numerous times for 129+ pollutants
Define geology primarily by driller's log & cuttings
Evaluate hydrology with water level maps only
Possibly obtain soil & core samples (chem. extractions)

BENEFITS

Rapid screening of problem
Moderate costs involved
Field and lab techniques standardized
Data analysis relatively straightforward
Tentative identification of remedial options possible

SHORTCOMINGS

True extent of problem often misunderstood
Selected remedial alternative may not be appropriate
Optimization of remedial actions not possible
Clean—up costs unpredictable and excessive
Verification of compliance uncertain and difficult

Table 5.2. State-of-the-Art Approach to Site Characterization

SITE CHARACTERIZATION
STATE–OF–THE–ART APPROACH

RECOMMENDED ACTIONS

Install depth–specific well clusters
Sample and analyze for 129+ pollutants initially
Analyze selected contaminants in subsequent samplings
Define geology by extensive coring / split–spoon samples
Evaluate hydrology with well clusters and geohydraulic tests
Perform limited tests on solids (grain size, clay content)
Conduct limited geophysical surveys (resistivity soundings)

BENEFITS

Conceptual understanding of problem more complete
Better prospect for optimization of remedial actions
Predictability of remediation effectiveness increased
Clean–up costs lowered, estimates improved
Verification of compliance more soundly based

SHORTCOMINGS

Characterization costs somewhat higher
Detailed understanding of problem still difficult
Full optimization of remedial actions not likely
Field tests may create secondary problems
Demand for specialists increased

Table 5.3. State-of-the-Science Approach to Site Characterization

SITE CHARACTERIZATION
STATE–OF–THE–SCIENCE APPROACH

IDEALIZED APPROACH

Assume 'State–of–the–Art Approach' as starting point
Conduct tracer–tests & borehole geophysical surveys
Determine % organic carbon, exchange capacity, etc. of solids
Measure redox potential, pH, dissolved oxygen, etc. of fluids
Evaluate sorption–desorption behavior using select cores
Identify bacteria & assess potential for biotransformation

BENEFITS

Thorough conceptual understanding of problem obtained
Full optimization of remedial actions possible
Predictability of remediation effectiveness maximized
Clean–up costs lowered significantly, estimates reliable
Verification of compliance assured

SHORTCOMINGS

Characterization costs significantly higher
Few previous field applications of advanced theories
Field and laboratory techniques not yet standardized
Availability of specialized equipment low
Demand for specialists dramatically increased

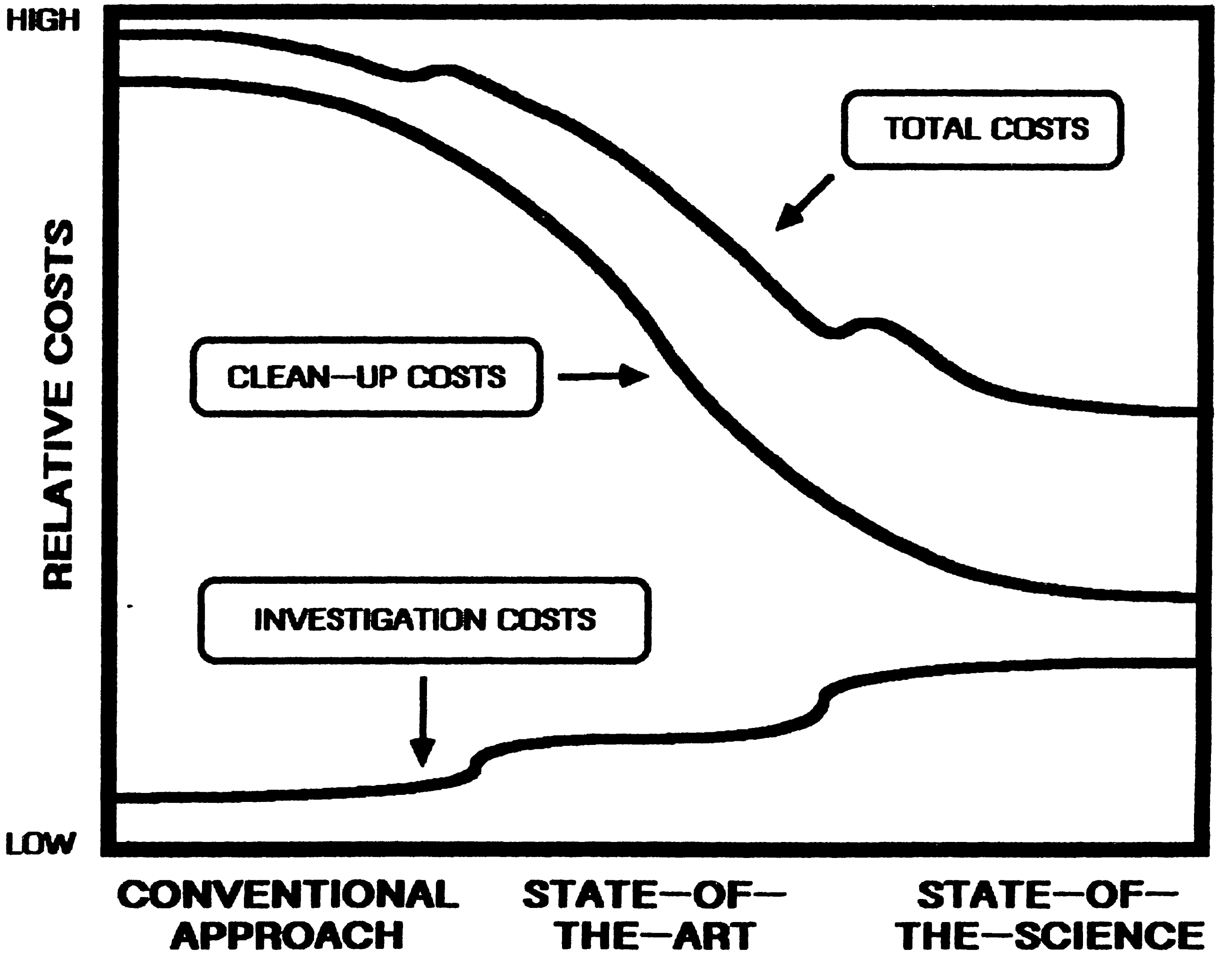


Figure 5.1. Conceptualization of Trade-Offs of Costs in Investigations and Clean-Ups, as a Function of Approach Used for Site Characterization

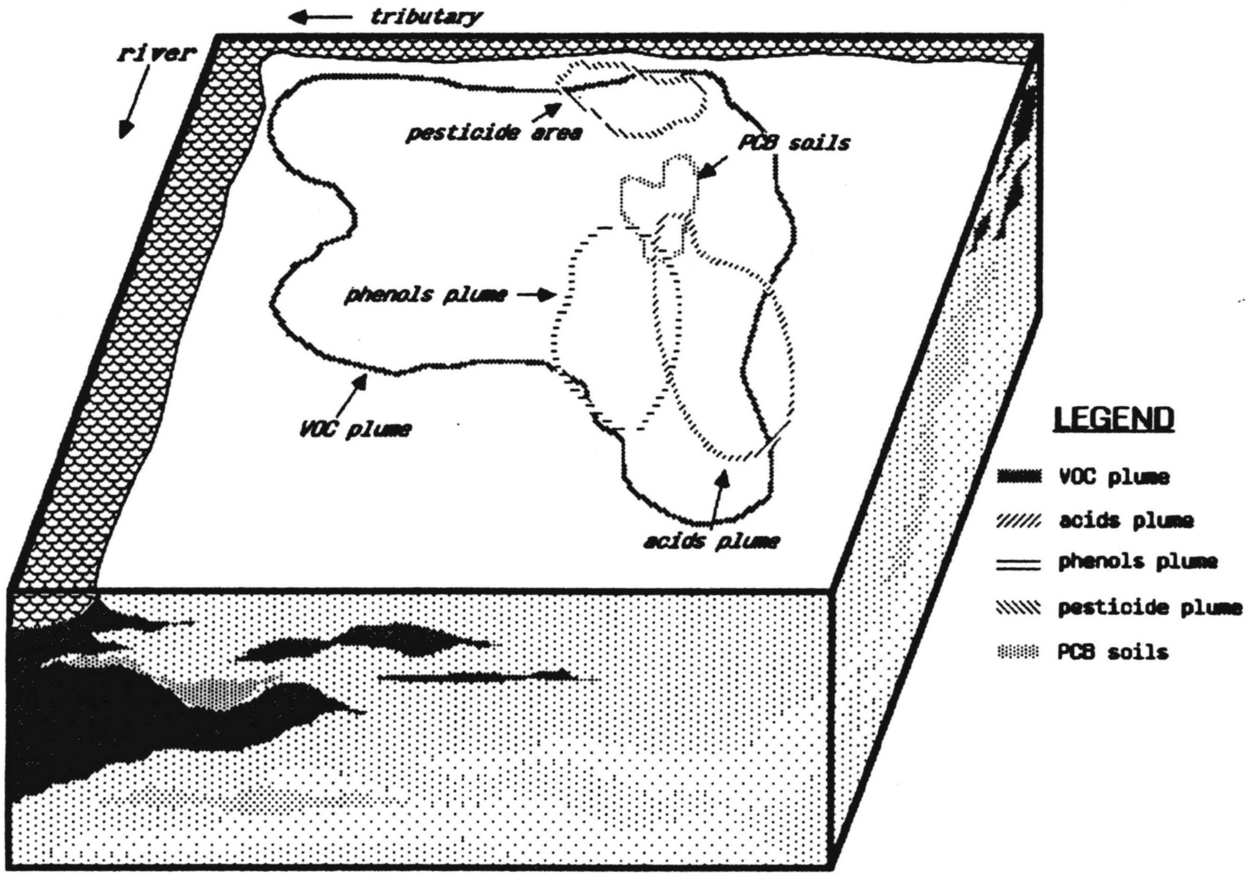


Figure 5.2. Hypothetical Ground-Water Contamination Problem

large amounts of pentachlorophenol), and a volatile organics plume (e.g., from solvent storge leaks). In addition, soils onsite are heavily contaminated in one area with spilled pesticides, and in another area with spilled transformer oils that contained PCB's in high concentrations.

The kind of clean-up that would likely result from a mediocre conventional site investigation is illustrated in Figure 5.3. The volatile organics plume would be considered to be the the most important to remediate, since it is the most mobile, and an extraction system would be installed. Extracted fluids would be air-stripped of volatiles and then passed through a treatment plant for removal of non-volatile residues, probably by relatively expensive filtration through granular activated carbon. Giving the benefit of doubt to the disigners, the areal locations and vertical positions of the extraction wells would be optimized according to information obtained from the drilling records of the monitoring wells. The PCB laden soils would be excavated and sent off to an incinerator or approved waste treatment and disposal facility.

The kind of remediation that would likely result from incorporating some state-of-the-art field methods in an investigation of this site is shown in Figure 5.4. There it can be seen that a special program to recover the acid plume and neutralize it would be instituted; the same would be true of the pesticide plume. This approach would probably lower treatment costs overall, despite the need for separate treatment trains for the different plumes, because substantially lesser amounts of non-volatile residues would need to be removed by expensive carbon filtration at the onsite treatment plant.

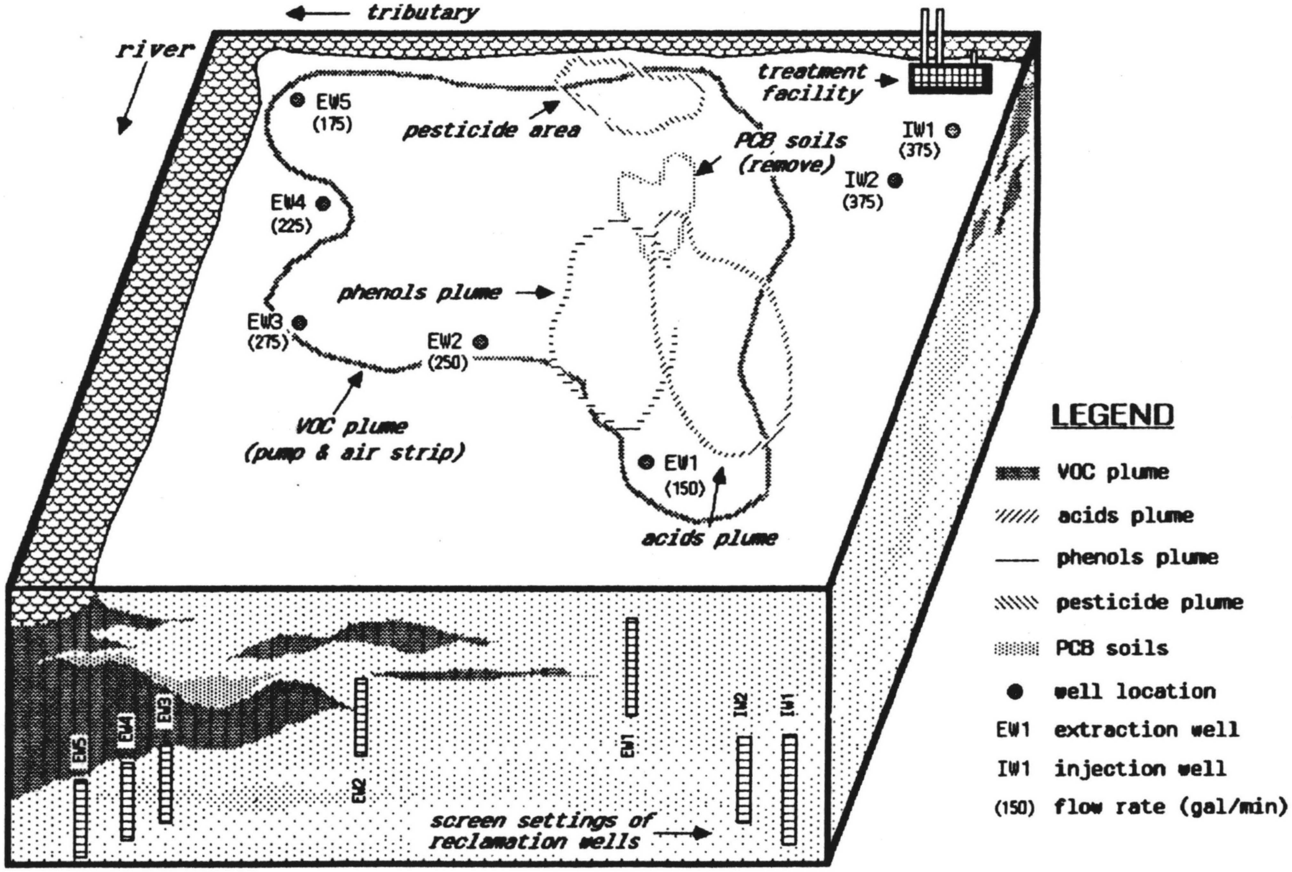


Figure 5.3. Typical Conventional Clean-Up Applied to Ground-Water Contamination Problem in Figure 5.2

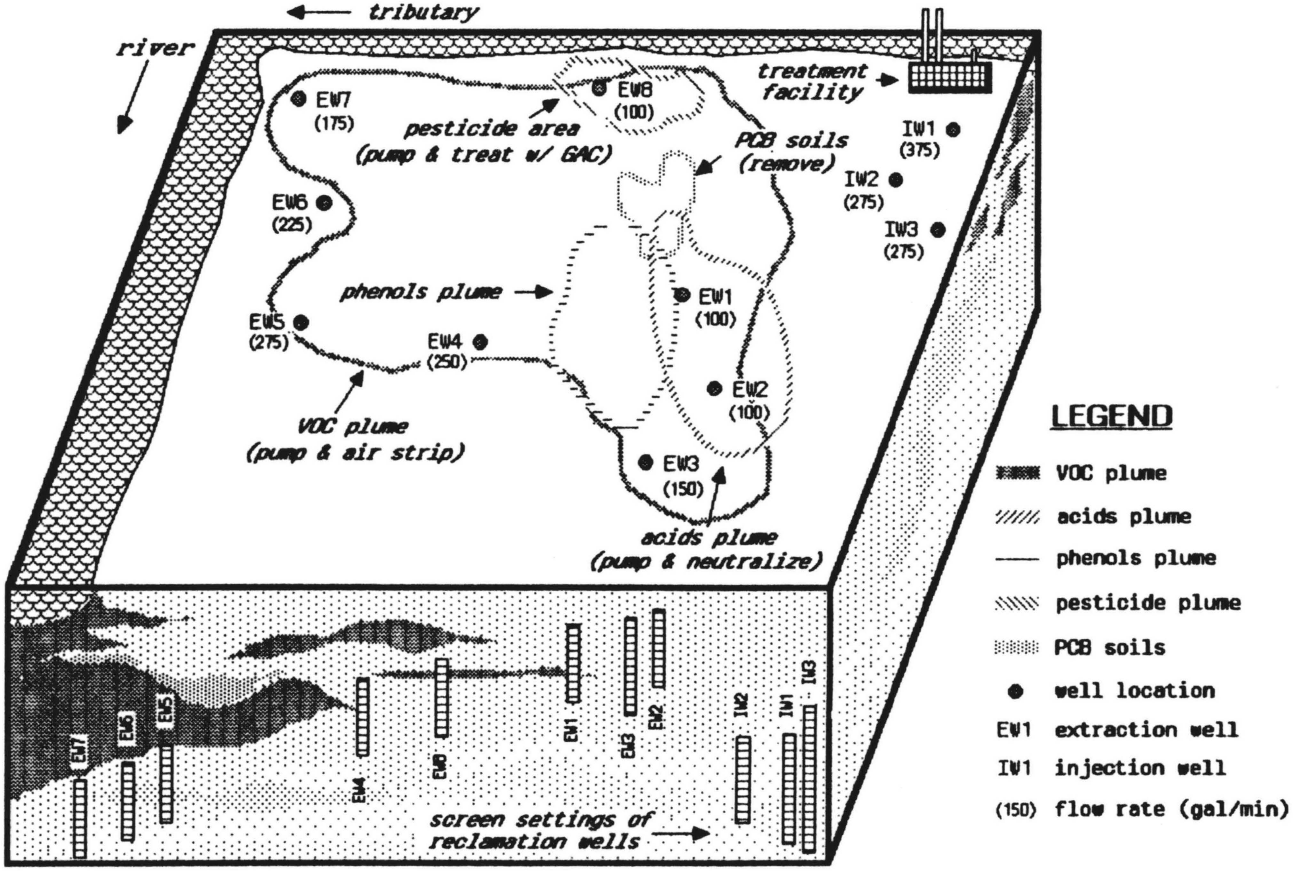


Figure 5.4. Moderate State-of-the-Art Clean-Up Applied to Ground-Water Contamination Problem in Figure 5.2

If all state-of-the-art investigation tools were used at the site, there would be an opportunity to evaluate the desireability of using a subsurface barrier wall to enhance remediation efforts (Figure 5.5). The wall would not be expected to entomb the plumes, but would serve the purpose of limiting pumping to contaminated fluids; rather than having the extracted waters diluted with fresh waters available to the extraction wells, as was true of the two previous approaches. The volume pumped would be lowered because the barrier wall would increase the drawdown at each well by hydraulic interference effects, thereby maintaining the same effective hydrodynamic control with lesser pumpage. Treatment costs would be lowered as well, because the waters pumped would contain higher concentrations of contaminants; treatment efficiencies normally fall with decreasing concentrations. Soil washing techniques would be used on the pesticide contaminated area to minimize future source releaces to ground water.

Finally, if state-of-the-science findings regarding potential biotransformations could be taken advantage of, it might be possible to effect in-situ degradation of the phenols plume, and remove volatile residues too (Figure 5.6).

5.2 Practical Guidance

The foregoing discussions should serve to signify the tremendous gains in efficiency that should be expected by better defining ground-water contamination problems and using that information to develop site-specific solutions, rather than implementing generalized solutions that have come into common practice out of a lack of recognition

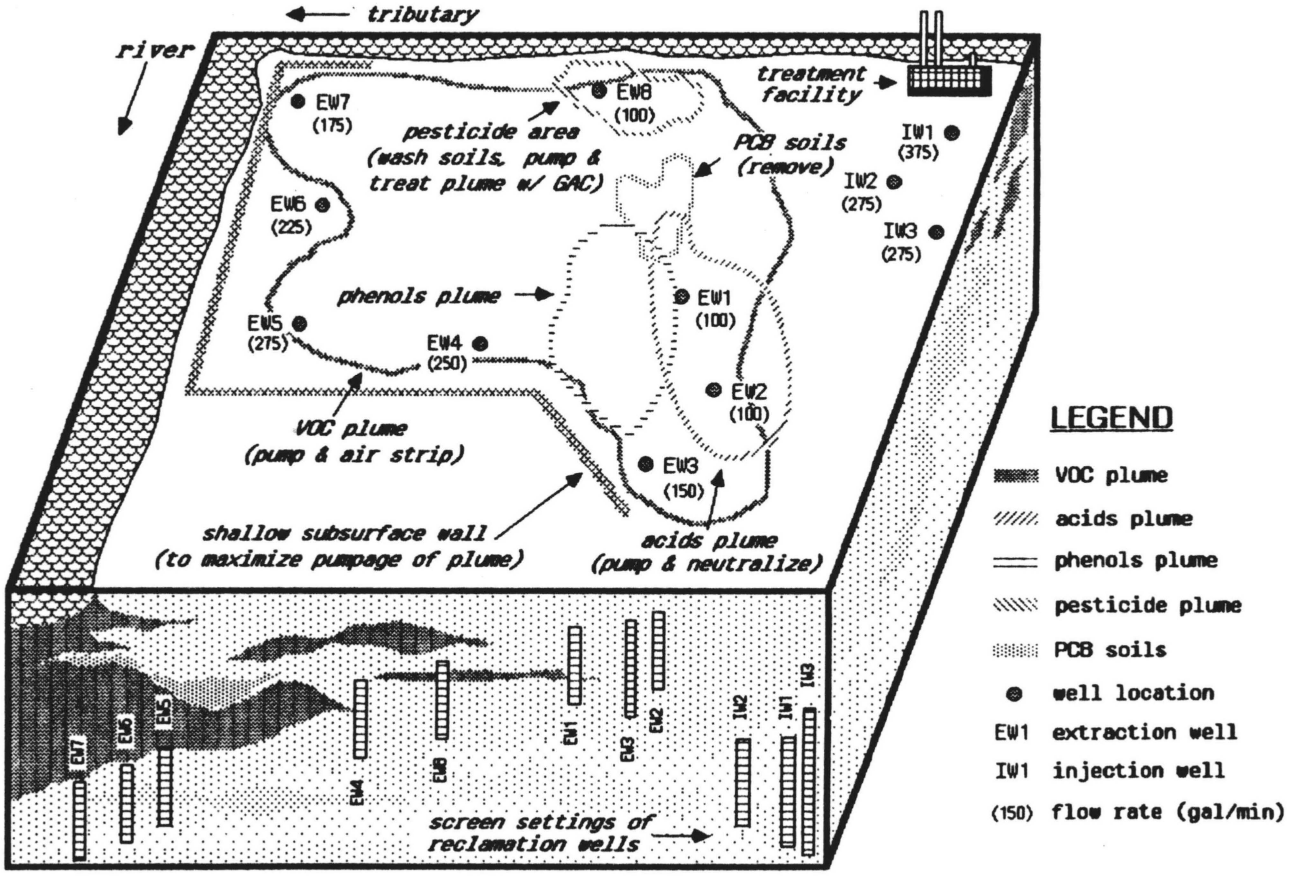


Figure 5.5. Advanced State-of-the-Art Clean-Up Applied to Ground-Water Contamination Problem in Figure 5.2

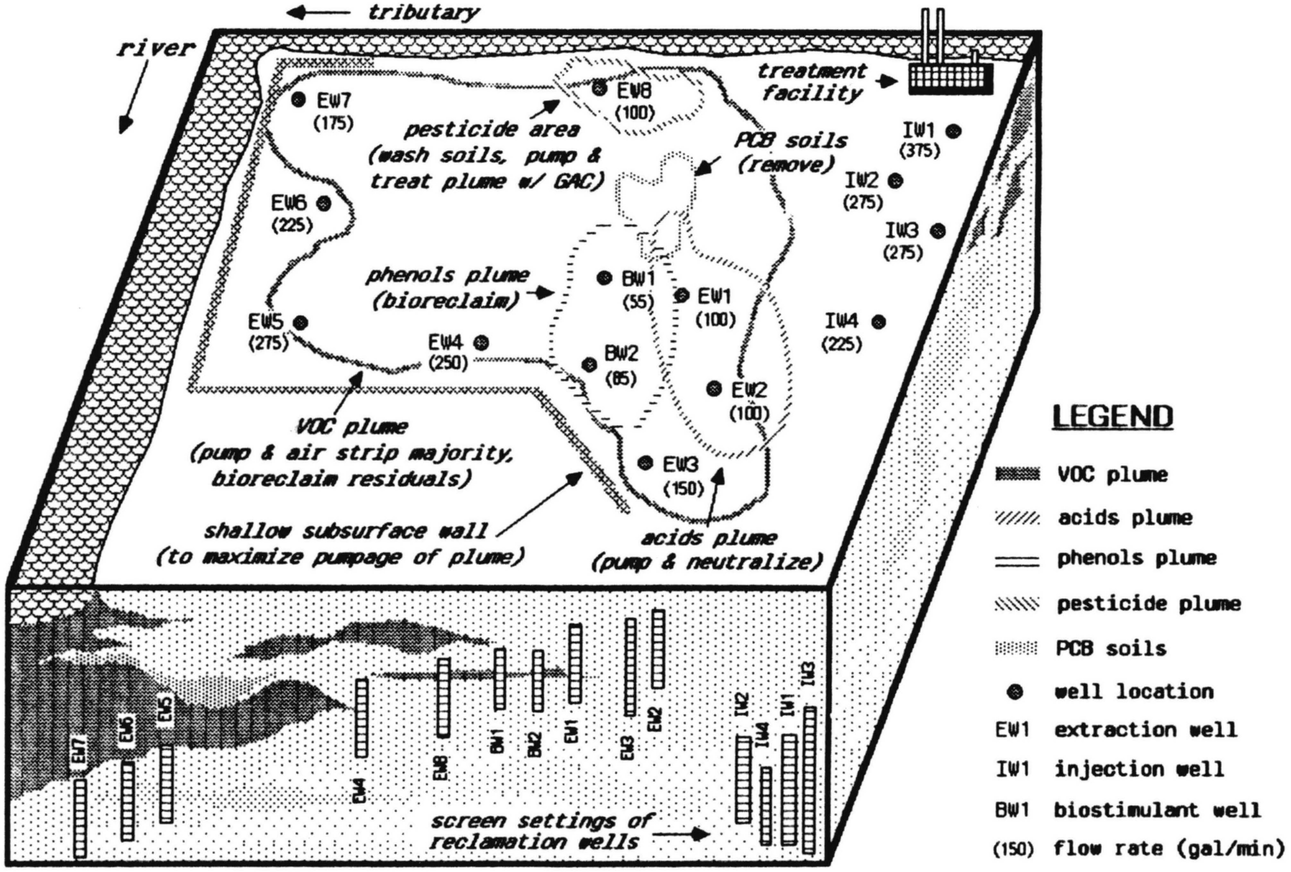


Figure 5.6. State-of-the-Science Clean-Up Applied to Ground-Water Contamination Problem in Figure 5.2

of the true economics involved. After all, this is not the first environmental issue where innovative technologies were expected to be far more expensive than later proved to be the case; Miller (1980) notes that the cost of controlling PVC exposure was overestimated by 200 times the actual amount.

5.2.1 Implications for Decisionmakers

Again, the point being made here is that current practices may be penny-wise and pound-foolish. If decisionmakers continue to take the stand that the appropriate strategy is to do the very least that can be done to minimize immediate threats, then it is likely that the same problem will be addressed over and over again. Simple economics will show that the cost of this policy of delaying truly complete remediations will be much higher over the long term than would be a policy of using the best approaches now; it just seems less painful to pay a small fee for interim solutions, because many sites can be touched on quickly and that public fears are temporarily quieted.

What this really means is that someone, somewhere is going to have to muster the political courage to say that it is not always possible to have our cake and eat it too. There will have to be a recognition of the limits that technology has, and a recognition of the up-front price to be paid for truly effective remediations. Most importantly, there will have to be a clear recognition of the fact that there will never be a magic solution found to waste disposal problems (Piasecki, 1984).

Everyone will have to pitch in, by separating their wastes into those that can be recycled, those that can be safely disposed of, those that can be economically treated, and, yes, those that cannot be safely disposed of or treated economically. The latter category does exist and hard decisions as which parts of the environment can -- no, must -- be written off, will need to be made. With current policies that dote on the notion that a technological solution will be found to these problems with moderately increased levels of funding (National Academy of Public Administration, 1984), the slow poisoning of hundreds of thousands of small (and, some not-so-small) parts of the environment is taking place under society's collective nose (Pye and Kelley, 1984).

5.2.2 Effective Communications

One of the principal problems underlying the current state of affairs is the poor level of communications between specialists and decisionmakers. That each group believes it is doing its job correctly is undoubtedly true. That the current state of communications often results in poorly focused efforts is also true. Ask the decisionmakers and they will say that it could all work better if technical specialists just spoke plain language, instead of jargon. It would also help if specialists would learn something of the political realities, they would add. After all, it may be best to do a bang-up job on each site, but there is only so much cash available this year and visible progress (e.g., removing waste drums from sites) must be made to quiet public fears; despite the fact that the invisible part of the problem (e.g., the contaminant plume) is the most threatening. The specialists, on the other hand, would argue that decisionmakers need to become better educated in science and technology so that they could appreciate the subtleties involved at each site. Besides, they receive sound scientific advice; why don't they take it?

The truth of the matter is that both sides have a long way to go; and both are guilty of very poor communication with the general public. Scientists do little to dispel the notion that additional research and site-specific funds will allow completely satisfactory resolution of any and all problems. Many of them actually seem to believe that. Decisionmakers are all too eager to sweep the public communication problem under the rug, too; stepping forward could cost them dearly in a political sense (remember President Carter's 'Malaise Speech'?). Frankly, the public is much more able to accept difficult situations than most decisionmakers would like to admit, because they don't want to be the first to step in troubled waters. Thus, the problem of effective communication between specialists and decisionmakers, and between these two groups and the public is not a simple matter.

Perhaps naively, a few tables of suggested questions to be asked by the two groups of one another are presented here (Tables 5.4, 5.5, and 5.6), in the hopes that these could be used to stimulate more effective dialogues. The same tables should be studied for insights as to how to present material for public consumption; both in terms of clarity, and in terms of honest appraisals of the costs and limitations that will need to be accepted.

5.3 Recommendations for Research

The greatest challenges in ground-water contaminant assessments lie in two complementary areas: improved field techniques to characterize

Table 5.4. Screening Level Questions to Help Focus Ground-Water Contamination Assessments

I. General Problem Definition

(1) What are the key issues; quantity, quality, or both?

(2) What are the controlling geologic, hydrologic, chemical and biological features?

(3) Are there reliable data (proper field scale, quality controlled, etc.) for preliminary assessments?

(4) Do we have the model(s) needed for appropriate simulations?

II. Initial Responses Needed

(1) What is the time-frame for action (imminent or long-term)?

(2) What actions, if taken now, can significantly delay the projected impacts?

(3) To what degree can mathematical simulations yield meaningful results for the action alternatives, given available data?

(4) What other techniques or information (generic models, past experience, etc.) would be useful for initial estimates?

III. Strategies for Further Study

(1) Are the critical data gaps identified; if not, how well can the specific data needs be determined?

(2) What are the trade-offs between additional data and increased certainty of the assessments?

(3) How much additional manpower and resources are necessary to improve mathematical modeling efforts?

(4) How long will it take to produce useful simulations, including quality control and error-estimation efforts?

Table 5.5. Conceptualization Questions to Help Focus Ground-Water Contamination Assessments

I. Field Techniques and Data Production

(1) Are the installation and sampling techniques that are to be used accepted? innovative? controversial?

(2) Where are the weak spots in the assessment, and can these be further minimized or eliminated?

(3) What are the limitations of field tests that estimate the natural processes parameters of this problem?

II. Model Input Parameters and Boundary Conditions

(1) How reliable are the estimates of the input parameters; are they quantified within accepted statistical bounds?

(2) What are the boundary conditions, and why are they appropriate to this problem?

(3) Have the initial conditions with which the model is calibrated been checked for accuracy and consistency?

(4) Are the spatial grid design(s) and time-steps of the model optimized for this problem?

III. Model Quality Control and Error Estimation

(1) Have these models been mathematically validated against other solutions to this kind of problem?

(2) Has anyone field verified these models before, by direct applications or simulation of controlled experiments?

(3) How do these models compare with others in terms of computational efficiency, ease of use, or modification?

(4) What special measures are being taken to estimate the overall errors of the simulations?

Table 5.6. Sociopolitical Questions to Help Focus Ground-Water Contamination Assessments

I. Demographic Considerations

(1) Is there a larger population endangered by the problem than we are able to provide sufficient responses to?

(2) Is it possible to present this assessment in both non-technical technical formats, to reach all audiences?

(3) What role can modeling play in public information efforts (e.g., effective graphics)?

(4) How prepared are we to respond to criticism of this assessment (e.g., supportive materials)?

II. Political Constraints

(1) Are there non-technical barriers to the techniques to be used to produce this assessment, such as 'tainted by association' with a controversy elsewhere?

(2) Do we have the cooperation of all involved parties in obtaining the necessary data and implementing solutions?

(3) Are similar technical efforts for this problem being undertaken by friend or foe?

(4) Can the results of the assessment be turned against us; are the results ambiguous or equivocal?

III. Legal Concerns

(1) Will these activities meet all regulations?

(2) If we are dependent on others for key inputs how do we recoup losses stemming from possible non-performance?

(3) What liabilities are incurred for projections arising from poor data, misinterpretations, or models used?

(4) Do any of the issues to be addressed by this assessment require the advice of attorneys?

natural processes, and theoretical developments to understand the interactions of physical, chemical, and biological processes.

5.3.1 Physical Processes Research

In the physical arena, there is a great need for test methods that can give unambiguous, representative estimates of key aquifer parameters. Hence, this is an area that should receive immediate attention. Other phenomena that need to be thoroughly investigated are immiscible and density flow behaviors; but, first things first -- there is still no satisfactory method for the rapid, accurate determination of the direction and rate of ground-water flow.

5.3.2 Chemical Processes Research

On the chemical side of things there are so many kinds of processes possible that it is difficult to recommend one over another. The current emphasis on sorption is probably justified, however, as this seems to explain a great deal of the behavior of the most troublesome contaminants. The role that co-solvation may play in shifting soprtion behavior is a crucial one, and one that is just now coming into its own. The role that the movement of fine particles may play in accelerating the movement of highly sorbed contaminants is just as crucial, though, and research in this area is as yet only conceptual.

5.3.3 Biological Processes Research

Rapid field methods to determine if biological adaptation to specific contaminants has occurred at a site are needed. Tools to predict whether adaptation can be expected, and to estimate the time required for adaptation if it does occur, are also necessary. For systems that are limited by transport processes, field methods to estimate the aquifer processes that control mixing, such as transverse dispersion and exchange processes across the water table, would be very helpful. For systems that are limited by the intrinsic biotransformation rate, new laboratory testing methods (possibly, improved microcosms) that will provide reliable estimates of the kinetic parameters are required.

5.3.4 General Comments

In addition to being sufficiently accurate and precise, new field methods should provide estimates that are truly representative of the hydrologic unit being simulated. Contaminants typically have long residence times in aquifers, and slow chemical and biological transformation rates can have environmental significance. Finally, there is a need for models that go beyond simple prediction of contaminant concentrations at some point in the aquifer, and forecast the concentrations in waters delivered by production wells.

5.4 Summary and Conclusions

This work has presented the status of ground-water contamination assessments as they are and as they might be. There is a large difference between what has been proven in a theoretical context and what is put into practice. Some of the reasons for this are that field methods to characterize important natural process parameters are still relatively crude, there remains an unwarranted perception that mathematical models can always back out the important parameters, and decisionmakers are reluctant to fund the use of state-of-the-art technologies that might avoid abuses of such models. The cost-effectiveness of conventional and state-of-the-art approaches to characterizing and remediating ground-water contamination problems need to be evaluated in terms of overall costs in the long term -- not up-front costs in the short term.

It is neither necessary nor appropriate to try to solve most ground-water contamination problems with the most sophisticated mathematical models available. The lack of adequate field tests to characterize natural processes affecting contaminant transport at most sites, together with the inherent variability of chemical data, make rigorous model calibration efforts little more than video arcade games. That is not to say that sophisticated mathematical models cannot be used to gain insights as to potential behavior of a plume; but when decisions are predicated solely on model outputs, the use has turned to abuse.

There will always be a significant degree of professional judgement called for in ground-water contamination assessments, and judging from the frequency with which significant errors are introduced by poor field work, this is an area that needs much attention. More strict licensing of engineers or scientists engaged in this kind of work will probably do little to improve the situation. Rather, much more effective communication between specialists and decisionmakers, and between these two groups and the public, is needed. Bringing the science out of the mystic realm into the harsh reality of the political arena, and demanding the political courage to describe and address the problem as it is -- with all its costs and technical limitations -- is the real solution.

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APPENDIXES

APPENDIX A

SOURCES OF INFORMATION ABOUT GROUND-WATER CONTAMINATION INVESTIGATIONS

SOURCES OF INFORMATION ABOUT GROUND-WATER CONTAMINATION INVESTIGATIONS

SOLID AND HAZARDOUS WASTE AGENCIES

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Pouch O
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AMERICAN SAMOA

Pati Faiai, Executive Secretary
Environmental Quality Commission
American Samoa Government
Pago Pago, American Samoa 96799
phone: Overseas Operator 633-4116

Randy Morris, Deputy Director
Department of Public Works
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ARIZONA

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APPENDIX B

BASIC LANGUAGE COMPUTER CODES FOR CAPTURE ZONE AND VELOCITY PLOT COMPUTATIONS

CAPZONE/VELDSTR PROGRAM

The CAPZONE/VELDSTR (CAPture ZONE / VELocity DiSTRibution) program is a BASIC language code that computes:

(1) regional velocities from field data (e.g., hydraulic conductivity, hydraulic gradient, and effective porosity),

(2) the dimensions of the capture zone of a single pumping well (specifically, the distance to the downgradient stagnation point and the maximum width upgradient), and

(3) the net velocity vector characteristics at an arbitrary number of observation points for an arbitrary number of pumping and injection wells; outputs include the X- and Y-components, the magnitude, and the compass bearing of the velocity vector (in degrees).

The code is presented here as it was developed on a Sharp PC-1500™ handheld microcomputer. Prompts appear on the liquid crystal display of the unit and a hardcopy is output to the paper-tape printer. A hardcopy of the code is given on the following pages (using CSIZE 2 as the printing font), obtained by entering the LLIST command after uploading the code into the program memory core of the PC-1500™. Several examples of the simulations possible with this code appear in APPENDIX C.

Figure B.1. CAPZONE/VELDSTR Program: Typed Version

```
10:WAIT 50:PRINT "CAP.ZONE/VEL.DSTR.-J.KEELY"

20:LF 5:CSIZE 5:COLOR 1

30:LPRINT "CAPZONEVELDSTRPROGRAM"

40:CSIZE 3:COLOR 2:LPRINT "by J.F.KEELY"

50:DIM XWELL(50),YWELL(50),FLRT(50)

60:DIM XOBS(50),YOBS(50)
```

```
70:DIM WXVEL(50),WYVEL(50)

80:CSIZE 2:COLOR 0:LF 3:CALCINDEX=0:DEGREE

85:CALCINDEX=CALCINDEX+1:LPRINT "CALCULATION NO.";CALCINDEX:LF 3

89:Y=1:N=0

90:INPUT "IS REG. VEL. KNOWN?(YorN) ";CHOICE

100:IF CHOICE=1THEN GOTO 200

110:LPRINT " REG. VEL. INPUTS ":GRAPH :LINE (0,0)-(216,50),2,3,
    B:TEXT :LF 2:COLOR 0

120:INPUT "HYD.COND.(GPD/SQ.FT.)? ";HYDCND

125:LPRINT HYD.COND(GPD/FT2)=",HYDCND

126:HUSA=HYDCND/7.48

127:LPRINT " (or in FT/DAY)=",HUSA

128:HMTR=(HYDCND/7.48)*30.48

129:LPRINT " (or in CM/DAY)=",HMTR

130:INPUT "HYD.GRAD.(FT/FT)? ";GRDNT

135:LPRINT "HYD.GRAD(FT/FT)=",GRDNT

140:INPUT "EFF.POROSITY(DECIMAL)? ";PRSTY

145:LPRINT "EFF.POROSITY(DEC)=",PRSTY

150:REGVEL=(HYDCND/7.48)*GRDNT/PRSTY

160:LF 1:CSIZE 3:COLOR 2

165:LPRINT "REG.VELOCITY (FT/DAY)":CSIZE 2:COLOR 0:LPRINT "
   ";REGVEL:LF 1

166:CSIZE 3:COLOR 2:RVMTR=REGVEL*30.48

167:LPRINT " (CM/DAY)":CSIZE 2:COLOR 0:LPRINT " ";RVMTR:LF 1

170:CSIZE 3:COLOR 2:LPRINT "<<<<<<>>>>>>":COLOR 0:CSIZE 2:LF 4

199:V=1:C=0

200:INPUT "C-ZONE or V-DSTR.?(CorV) ";CZVD
```

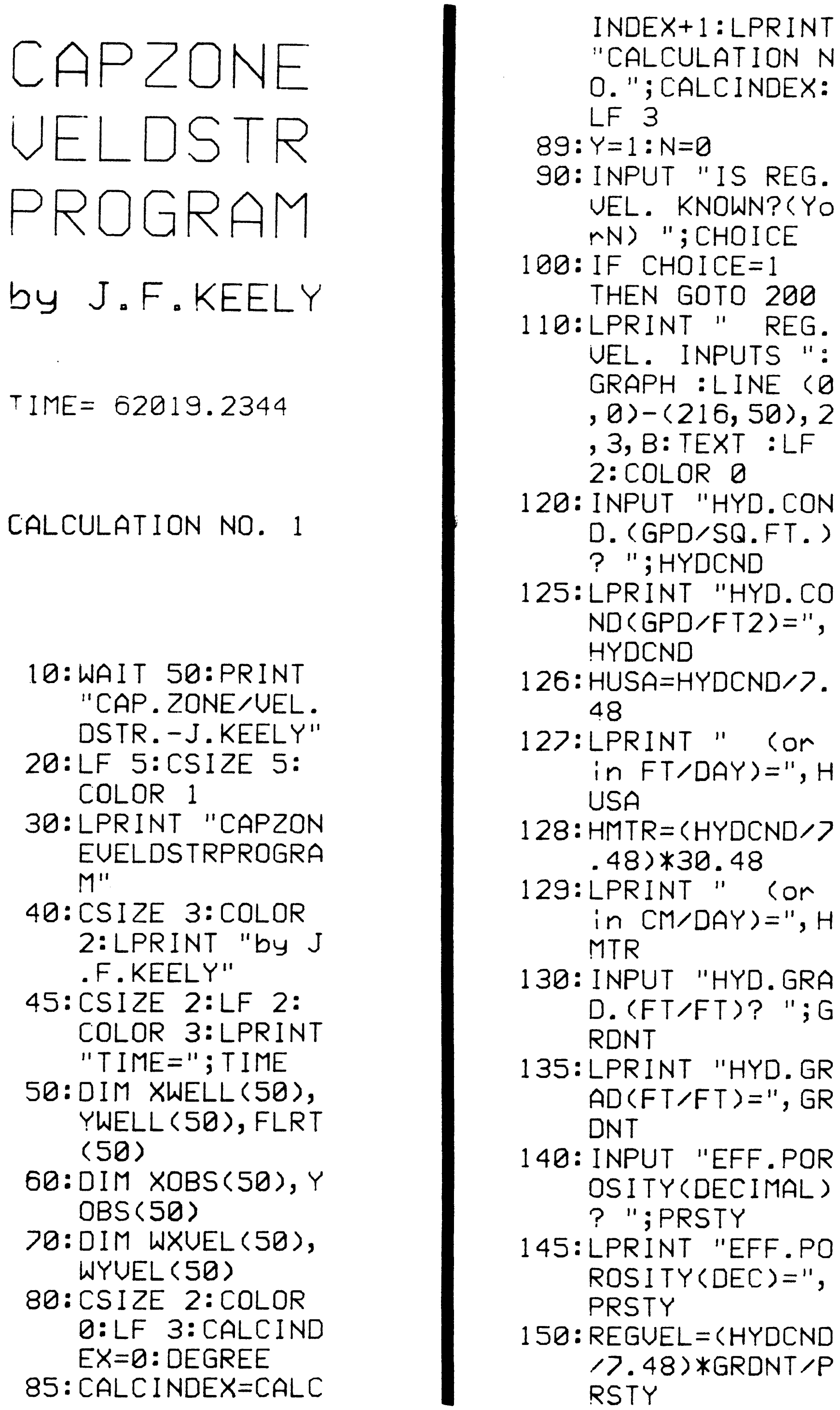
```
210:IF CZVD=1THEN GOTO 700
220:LPRINT " CAPT.ZONE INPUTS ":GRAPH :LINE (0,0)-(216,50),2,3,B:TEXT
   :LF 2:COLOR 0
230:INPUT "REG.VEL.(FT/DAY)? ";REGVEL
240:LPRINT "REG.VEL.(FT/DAY)=",REGVEL
250:INPUT "EFF.SAT.THICK.(FT)? ";SATHK
255:LPRINT "EFF.SAT.THICK(FT)=",SATHK
260:INPUT "EFF.POROSITY(DECIMAL)? ";PRSTY
265:LPRINT "EFF.POROSITY(DEC)=",PRSTY
270:INPUT "FLOW RATE(GPM)? ";FLRT
275:LPRINT "FLOW RATE(GPM)=",FLRT
280:LF 1:CSIZE 3:COLOR 2
285:LPRINT "CAPTURE ZONE DIMENSIONS ":CSIZE 2:COLOR 0
290:STAGPTDWNGR=(FLRT*1440/7.48)/(2*π*SATHK*PRSTY*REGVEL)
295:LPRINT " STAGNATION POINT":LPRINT " DOWNGRADIENT(FT)",STAGPTDWNGR
300:UPGRCPZNWDTH=2*π*STAGPTDWNGR
305:LPRINT "MAXIMUM UPGRADIENT":LPRINT "CAP.ZONE
    WIDTH(FT)",UPGRCPZNWDTH
306:LF 1:CSIZE 3:COLOR 2:LPRINT "<<<<<<>>>>>>":COLOR 0:CSIZE 2:LF 4
309:Y=1:N=0
310:INPUT "MORE CALCULATIONS?(YorN) ";AGAIN
320:IF AGAIN=1THEN GOTO 85
330:LF 2:CSIZE 4:COLOR 1
340:LPRINT "PROGRAM OVER, THANK YOU":LF 3:COLOR 0:CSIZE 2:END
700:CSIZE 2:COLOR 0:LPRINT "VEL.DSTR. INPUTS"
705:GRAPH :LINE (0,0)-(216,50),2,3,B:TEXT :COLOR 0:LF 2
710:INPUT "REG.VEL.(FT/DAY)? ";REGVEL
715:LPRINT "REG.VEL.(FT/DAY)=",REGVEL
```

```
720:INPUT "REG.FLOW DIR.(N=0,E=90)?";THETA

725:LPRINT"REG.FLOW DIRECTION":LPRINT "(COMPASS DEGREES)=",THETA

730:INPUT "EFF.SAT.THICK.(FT)? ";SATHK

735:LPRINT "EFF.SAT.THICK(FT)=",SATHK

740:INPUT "EFF.POROSITY(DECIMAL)? ";PRSTY

745:LPRINT "EFF.POROSITY(DEC)=",PRSTY

750:INPUT "# OF WELLS(MAX=50)?";W:LF 2

755:FOR J=1TO W:CSIZE 3:COLOR 2

757:LPRINT " WELL NO.";J:CSIZE 2:COLOR 0

760:INPUT "FLOW(GPM:+PUMP,-INJ)? ";FLRT(J)

765:LPRINT "FLOW RATE(GPM)=",FLRT(J)

770:INPUT "WELL X-COORD.(FT)? ";XWELL(J)

775:LPRINT "WELL X-COORD.(FT)=",XWELL(J)

780:INPUT "WELL Y-COORD.(FT)? ";YWELL(J)

785:LPRINT "WELL Y-COORD.(FT)=",YWELL(J):LF 1

790:NEXT J

800:INPUT "# OF OBS. POINTS(MAX=50)?";N:LF 2

810:FOR I=1TO N:CSIZE 3:COLOR 2

815:LPRINT "OBS.POINT";I:CSIZE 2:COLOR 0

820:INPUT "X-COORD.(FT)? ";XOBS(I)

825:LPRINT "X-COORD.(FT)=";XOBS(I)

830:INPUT "Y-COORD.(FT)? ";YOBS(I)

835:LPRINT "Y-COORD.(FT)=";YOBS(I):LF 1

836:CSIZE 3:COLOR 2:LPRINT "<<<<<<>>>>>>":COLOR 0:CSIZE 2

838:WXVEL=0:WYVEL=0

840:FOR J=1TO W
```

```
845:A=XOBS(I)-XWELL(J):B=YOBS(I)-YWELL(J)

850:WXVEL(J)=((FLRT(J)*1440/7.48)/(2*π*SATHK*PRSTY))*(A/(A^2+B^2))

851:WYVEL(J)=((FLRT(J)*1440/7.48)/(2*π*SATHK*PRSTY))*(B/(A^2+B^2))

856:WXVEL=WXVEL+WXVEL(J)

857:WYVEL=WYVEL+WYVEL(J)

858:NEXT J

860:XVEL=(REGVEL*SIN THETA)-WXVEL

861:YVEL=(REGVEL*COS THETA)-WYVEL

862:LPRINT "VEL. X-COMPONENT":LPRINT "(FT/DAY)=",XVEL

863:LPRINT "VEL. Y-COMPONENT":LPRINT "(FT/DAY)=",YVEL

865:NETVEL=√(XVEL^2+YVEL^2)

866:LPRINT "NET VEL.(FT/DAY)=",NETVEL

870:IF YVEL=0THEN GOTO 873

872:GOTO 885

873:IF SGN XVEL=+1THEN GOTO 883

874:BETA=THETA+180

875:IF BETA>360THEN GOTO 878

876:GOTO 895

878:BETA=BETA-360: GOTO 895

883:BETA=THETA: GOTO 895

885:BETA=ATN (XVEL/YVEL)

886:IF SGN YVEL=-1THEN GOTO 893

887:IF SGN BETA=-1THEN GOTO 890

888:GOTO 895

890:BETA=BETA+360:GOTO 895

893:BETA=BETA+180

895:LPRINT "NET FLOW DIRECTION":LPRINT "(COMPASS DEGREES)=",BETA:LF 1
```

```
896:CSIZE 3:COLOR 2:LPRINT "<<<<<<>>>>>>":COLOR 0:CSIZE 2

900:LF 1:NEXT I

909:Y=1:N=0

910:INPUT "MORE CALCULATIONS?(YorN) ";AGAIN

920:IF AGAIN=1THEN GOTO 85

930:LF 2:CSIZE 4:COLOR 1

940:LPRINT " PROGRAM OVER, THANK YOU":LF 3:COLOR 0:CSIZE 2:END
```

Figure B.2. CAPZONE/VELDSTR Program: Computer Generated Version



```
CAPZONE
VELDSTR
PROGRAM

by J.F.KEELY


TIME= 62019.2344


CALCULATION NO. 1



 10:WAIT 50:PRINT
    "CAP.ZONE/VEL.
    DSTR.-J.KEELY"
 20:LF 5:CSIZE 5:
    COLOR 1
 30:LPRINT "CAPZON
    EVELDSTRPROGRA
    M"
 40:CSIZE 3:COLOR
    2:LPRINT "by J
    .F.KEELY"
 45:CSIZE 2:LF 2:
    COLOR 3:LPRINT
    "TIME=";TIME
 50:DIM XWELL(50),
    YWELL(50),FLRT
    (50)
 60:DIM XOBS(50),Y
    OBS(50)
 70:DIM WXVEL(50),
    WYVEL(50)
 80:CSIZE 2:COLOR
    0:LF 3:CALCIND
    EX=0:DEGREE
 85:CALCINDEX=CALC
    INDEX+1:LPRINT
    "CALCULATION N
    O.";CALCINDEX:
    LF 3
 89:Y=1:N=0
 90:INPUT "IS REG.
    VEL. KNOWN?(Yo
    rN) ";CHOICE
100:IF CHOICE=1
    THEN GOTO 200
110:LPRINT "  REG.
    VEL. INPUTS ":
    GRAPH :LINE (0
    ,0)-(216,50),2
    ,3,B:TEXT :LF
    2:COLOR 0
120:INPUT "HYD.CON
    D.(GPD/SQ.FT.)
    ? ";HYDCND
125:LPRINT "HYD.CO
    ND(GPD/FT2)=",
    HYDCND
126:HUSA=HYDCND/7.
    48
127:LPRINT "  (or
    in FT/DAY)=",H
    USA
128:HMTR=(HYDCND/7
    .48)*30.48
129:LPRINT "  (or
    in CM/DAY)=",H
    MTR
130:INPUT "HYD.GRA
    D.(FT/FT)? ";G
    RDNT
135:LPRINT "HYD.GR
    AD(FT/FT)=",GR
    DNT
140:INPUT "EFF.POR
    OSITY(DECIMAL)
    ? ";PRSTY
145:LPRINT "EFF.PO
    ROSITY(DEC)=",
    PRSTY
150:REGVEL=(HYDCND
    /7.48)*GRDNT/P
    RSTY
```

```
160:LF 1:CSIZE 3:
    COLOR 2
165:LPRINT "REG.VE
    LOCITY  (FT/DA
    Y)":CSIZE 2:
    COLOR 0:LPRINT
    "  ";REGVEL:LF
    1
166:CSIZE 3:COLOR
    2:RVMTR=REGVEL
    *30.48
167:LPRINT "  (CM/
    DAY)":CSIZE 2:
    COLOR 0:LPRINT
    "  ";RVMTR:LF
    1
170:CSIZE 3:COLOR
    2:LPRINT "<<<<
    <<>>>>>>":
    COLOR 0:CSIZE
    2:LF 4
199:V=1:C=0
200:INPUT "C-ZONE
    or V-DSTR.?(Co
    rV) ";CZVD
210:IF CZVD=1THEN
    GOTO 700
220:LPRINT " CAPT.
    ZONE INPUTS ":
    GRAPH :LINE (0
    ,0)-(216,50),2
    ,3,B:TEXT :LF
    2:COLOR 0
230:INPUT "REG.VEL
    .(FT/DAY)? ";R
    EGVEL
240:LPRINT "REG.VE
    L.(FT/DAY)=",R
    EGVEL
250:INPUT "EFF.SAT
    .THICK.(FT)? "
    ;SATHK
255:LPRINT "EFF.SA
    T.THICK(FT)=",
    SATHK
260:INPUT "EFF.POR
    OSITY(DECIMAL)
    ? ";PRSTY
265:LPRINT "EFF.PO
    ROSITY(DEC)=",
    PRSTY
270:INPUT "FLOW RA
    TE(GPM)? ";FLR
    T
275:LPRINT "FLOW R
    ATE(GPM)=",FLR
    T
280:LF 1:CSIZE 3:
    COLOR 2
285:LPRINT "CAPTUR
    E ZONE DIMENSI
    ONS ":CSIZE 2:
    COLOR 0
290:STAGPTDWNGR=(F
    LRT*1440/7.48)
    /(2*π*SATHK*PR
    STY*REGVEL)
295:LPRINT " STAGN
    ATION POINT":
    LPRINT " DOWNG
    RADIENT(FT)",S
    TAGPTDWNGR
300:UPGRCPZNWDTH=2
    *π*STAGPTDWNGR
305:LPRINT "MAXIMU
    M UPGRADIENT":
    LPRINT "CAP.ZO
    NE WIDTH(FT)",
    UPGRCPZNWDTH
306:LF 1:CSIZE 3:
    COLOR 2:LPRINT
    "<<<<<<>>>>>>"
    :COLOR 0:CSIZE
    2:LF 4
309:Y=1:N=0
310:INPUT "MORE CA
    LCULATIONS?(Yo
    rN) ";AGAIN
320:IF AGAIN=1THEN
    GOTO 85
330:LF 2:CSIZE 4:
    COLOR 1
340:LPRINT " PROGR
    AM   OVER,   TH
```

```
     ANK YOU":LF 3:
     COLOR 0:CSIZE
     2:END
700:CSIZE 2:COLOR
     0:LPRINT " VEL
     .DSTR. INPUTS"
705:GRAPH :LINE (0
     ,0)-(216,50),2
     ,3,B:TEXT :
     COLOR 0:LF 2
710:INPUT "REG.VEL
     .(FT/DAY)? ";R
     EGVEL
715:LPRINT "REG.VE
     L.(FT/DAY)=",R
     EGVEL
720:INPUT "REG.FLO
     W DIR.(N=0,E=9
     0)?";THETA
725:LPRINT "REG.FL
     OW DIRECTION":
     LPRINT "(COMPA
     SS DEGREES)=",
     THETA
730:INPUT "EFF.SAT
     .THICK.(FT)? "
     ;SATHK
735:LPRINT "EFF.SA
     T.THICK(FT)=",
     SATHK
740:INPUT "EFF.POR
     OSITY(DECIMAL)
     ? ";PRSTY
745:LPRINT "EFF.PO
     ROSITY(DEC)=",
     PRSTY
750:INPUT "# OF WE
     LLS(MAX=50)?";
     W:LF 2
755:FOR J=1TO W:
     CSIZE 3:COLOR
     2
757:LPRINT " WELL
     NO.";J:CSIZE 2
     :COLOR 0
760:INPUT "FLOW(GP
     M:+PUMP,-INJ)?
      ";FLRT(J)
765:LPRINT "FLOW R
     ATE(GPM)=",FLR
     T(J)
770:INPUT "WELL X-
     COORD.(FT)? ";
     XWELL(J)
775:LPRINT "WELL X
     -COORD.(FT)=",
     XWELL(J)
780:INPUT "WELL Y-
     COORD.(FT)? ";
     YWELL(J)
785:LPRINT "WELL Y
     -COORD.(FT)=",
     YWELL(J):LF 1
790:NEXT J
800:INPUT "# OF OB
     S.POINTS(MAX=5
     0)?";N:LF 2
810:FOR I=1TO N:
     CSIZE 3:COLOR
     2
815:LPRINT "OBS.PO
     INT";I:CSIZE 2
     :COLOR 0
820:INPUT "X-COORD
     .(FT)? ";XOBS(
     I)
825:LPRINT "X-COOR
     D.(FT)=";XOBS(
     I)
830:INPUT "Y-COORD
     .(FT)? ";YOBS(
     I)
835:LPRINT "Y-COOR
     D.(FT)=";YOBS(
     I):LF 1
836:CSIZE 3:COLOR
     2:LPRINT "<<<<
     <<>>>>>>":
     COLOR 0:CSIZE
     2
838:WXVEL=0:WYVEL=
     0
840:FOR J=1TO W
845:A=XOBS(I)-XWEL
```

```
    L(J):B=YOBS(I)
    -YWELL(J)
850:WXVEL(J)=((FLR
    T(J)*1440/7.48
    )/(2*π*SATHK*P
    RSTY))*(A/(A^2
    +B^2))
851:WYVEL(J)=((FLR
    T(J)*1440/7.48
    )/(2*π*SATHK*P
    RSTY))*(B/(A^2
    +B^2))
856:WXVEL=WXVEL+WX
    VEL(J)
857:WYVEL=WYVEL+WY
    VEL(J)
858:NEXT J
860:XVEL=(REGVEL*
    SIN THETA)-WXV
    EL
861:YVEL=(REGVEL*
    COS THETA)-WYV
    EL
862:LPRINT "VEL. X
    -COMPONENT":
    LPRINT "(FT/DA
    Y)=",XVEL
863:LPRINT "VEL. Y
    -COMPONENT":
    LPRINT "(FT/DA
    Y)=",YVEL
865:NETVEL=√(XVEL^
    2+YVEL^2)
866:LPRINT "NET VE
    L.(FT/DAY)=",N
    ETVEL
870:IF YVEL=0THEN
    GOTO 873
872:GOTO 885
873:IF SGN XVEL=+1
    THEN GOTO 883
874:BETA=THETA+180
875:IF BETA>360
    THEN GOTO 878
876:GOTO 895
878:BETA=BETA-360:
    GOTO 895
```

```
883:BETA=THETA:
    GOTO 895
885:BETA=ATN (XVEL
    /YVEL)
886:IF SGN YVEL=-1
    THEN GOTO 893
887:IF SGN BETA=-1
    THEN GOTO 890
888:GOTO 895
890:BETA=BETA+360:
    GOTO 895
893:BETA=BETA+180
895:LPRINT "NET FL
    OW DIRECTION":
    LPRINT "(COMPA
    SS DEGREES)=",
    BETA:LF 1
896:CSIZE 3:COLOR
    2:LPRINT "<<<<
    <<>>>>>>":
    COLOR 0:CSIZE
    2
900:LF 1:NEXT I
909:Y=1:N=0
910:INPUT "MORE CA
    LCULATIONS?(Yo
    rN) ";AGAIN
920:IF AGAIN=1THEN
    GOTO 85
930:LF 2:CSIZE 4:
    COLOR 1
940:LPRINT " PROGR
    AM  OVER,  TH
    ANK YOU":LF 3:
    COLOR 0:CSIZE
    2:END
```

```
 PROGRAM
  OVER,
THANK YOU
```

APPENDIX C

EXAMPLE SIMULATIONS OF CAPTURE ZONES, VELOCITY PLOTS, AND WELL HYDRAULICS

The CAPZONE/VELDSTR program is loaded into the memory of the Radio Shack PC-2 or the Sharp PC-1500 per the standard instructions for either keystroke entry, or loading from a cassette tape (e.g. CLOAD "CAPZONE/VELDSTR"). After loading into memory, the standard RUN command is entered by keystroke and program execution begins immediately with the program title displayed on the liquid crystal display (LCD) at the same time as it is being printed on the paper tape.

As soon as the title has been printed, a prompt appears on the LCD; REG.VEL.KNOWN? (Y or N). If the regional velocity is known, the user enters Y ('y' is not allowed); if not, the user enters N. If N was entered, a prompt appears on the LCD for the hydraulic conductivity of the aquifer in question; HYD.COND.(GPD/FT2)? The user enters the estimated value and the printer immediately echo's the same information; for convenience, equivalent values of hydraulic conductivity are computed in feet per day and centimeters per day, and also appear on the paper tape. A prompt then appears on the LCD for the hydraulic gradient, which the user responds to in kind. This process is repeated once more, for the effective porosity estimate. As soon as this is done, the regional velocity estimate is computed and is printed on the paper tape in both feet per day and centimeters per day.

After the regional velocity estimate is printed, a prompt appears on the LCD (the same prompt as would have appeared had Y been entered in response to REG.VEL.KNOWN? (Y or N); CAP.ZONE or VEL.DIST.? (C or V). If C is entered a series of prompts will appear on the LCD for the regional velocity, the effective saturated thickness, the effective porosity and the flow rate of the well in question. The user responds to each of these, the printer confirms the entries, and the dimensions of the capture zone (distance to downgradient stagnation point and maximum width of the capture zone upgradient) are printed on the paper tape. The program then asks the user if more calculations are desired; MORE CALCULATIONS? (Y or N). If Y is entered, the REG.VEL.KNOWN? (Y or N) prompt again appears. If N is entered, PROGRAM OVER, THANK YOU is printed on the paper tape and execution of the program ceases; the > prompt appears on the LCD, indicating the computer is ready for a new BASIC command (CAPZONE/VELDSTR remains in memory, but must be activated with another RUN command).

If V had been entered in response to CAP.ZONE or VEL.DIST.? (C or V), the velocity distribution part of the code would have been invoked. The user would have been prompted for the regional velocity, the direction of regional flow in compass bearing degrees (Figure C.1), the effective saturated thickness, the effective porosity, and the total number of pumping and injection wells involved. At this point, the computer would begin prompting for the flow rate and X- and Y-coordinates of each well (see Figure C.1 for the orientation of the X- and Y-coordinates, relative to the direction of regional flow). Pumping wells have positive flow rate values, injection wells have negative flow rates; if too many wells are accidentally asked for, entering a zero flow rate for a well is allowed and results in effectively removing it from the analysis. When the information for all wells has been entered, the computer will prompt for the number of observation points desired. It then sequentially prompts for the X- and Y-coordinates of the first observation point. As soon as the user has entered these, the X- and Y-components of the net velocity, the net velocity itself, and the net flow direction of the aquifer at that observation point are printed on the paper tape. This process is repeated for all observation points, after which the MORE CALCULATIONS? (Y or N) prompt appears on the LCD.

Typical runs of CAPZONE/VELDSTR are shown in Figures C.2 through C.5.

Figure C.1. CAPZONE/VELDSTR Program Axis Orientation Illustration

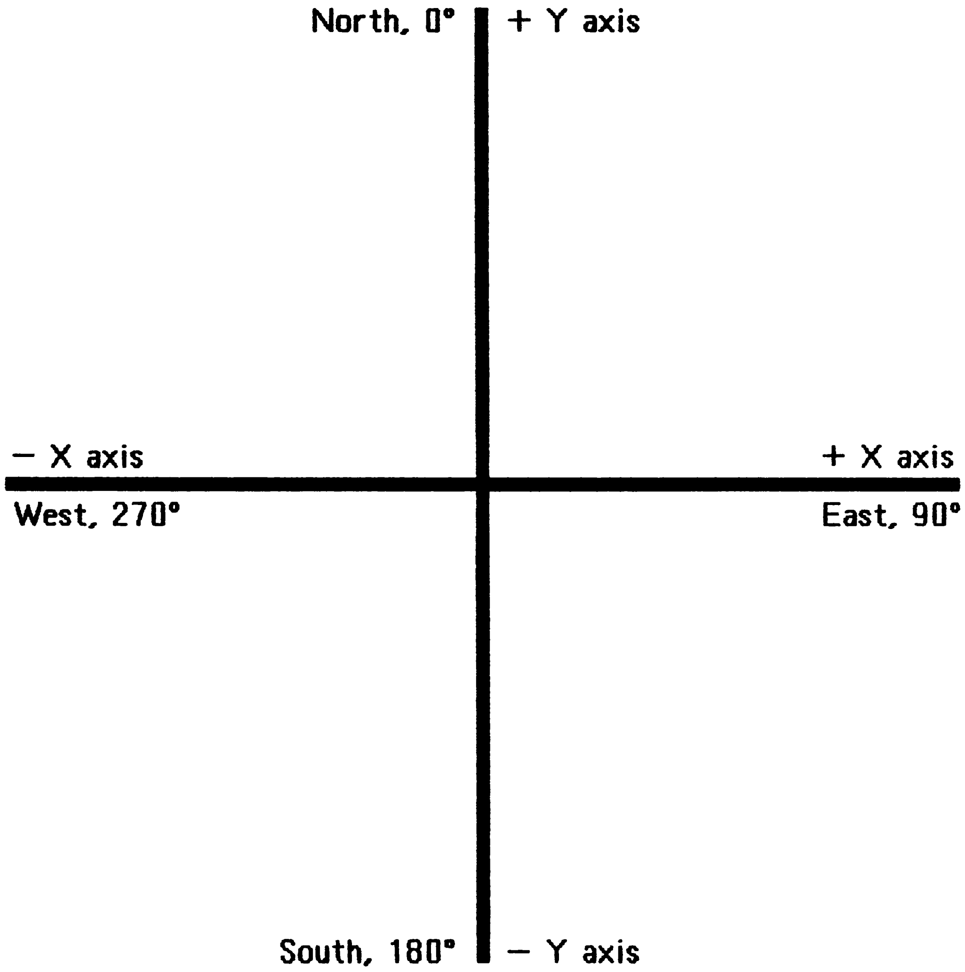


Figure C.2. CAPZONE/VELDSTR Program Simulation Example No.1

```
CAPZONE
VELDSTR
PROGRAM

by J.F.KEELY


TIME= 62112.4341


CALCULATION NO. 1


!----------------------------!
! REG.VEL.  INPUTS           !
!----------------------------!

HYD.COND(GPD/FT2)=
                   75
  (or in FT/DAY)=
       10.02673797
  (or in CM/DAY)=
       305.6149733
HYD.GRAD(FT/FT)=
                0.015
EFF.POROSITY(DEC)=
                 0.15

REG.VELOCITY
  (FT/DAY)
  1.002673797

  (CM/DAY)
  30.56149733

<<<<<<>>>>>>
```

```
!----------------------------!
! CAPT.ZONE  INPUTS          !
!----------------------------!

REG.VEL.(FT/DAY)=
       1.002673797
EFF.SAT.THICK(FT)=
                  100
EFF.POROSITY(DEC)=
                 0.15
FLOW RATE(GPM)=
                   50

CAPTURE ZONE
 DIMENSIONS

 STAGNATION POINT
 DOWNGRADIENT(FT)
      101.8591636
MAXIMUM UPGRADIENT
CAP.ZONE WIDTH(FT)
      640.0000001

<<<<<<>>>>>>



CALCULATION NO. 2


!----------------------------!
! CAPT.ZONE  INPUTS          !
!----------------------------!

REG.VEL.(FT/DAY)=
                0.001
EFF.SAT.THICK(FT)=
                  100
EFF.POROSITY(DEC)=
                 0.15
FLOW RATE(GPM)=
                   50

CAPTURE ZONE
 DIMENSIONS
```

```
 STAGNATION POINT
 DOWNGRADIENT(FT)
       102131.5143
MAXIMUM UPGRADIENT
CAP.ZONE WIDTH(FT)
         641711.23

<<<<<<>>>>>>



CALCULATION NO. 3

+--------------------+
| CAPT.ZONE  INPUTS  |
+--------------------+

REG.VEL.(FT/DAY)=
       1.002673797
EFF.SAT.THICK(FT)=
               100
EFF.POROSITY(DEC)=
              0.15
FLOW RATE(GPM)=
               -50

CAPTURE  ZONE
  DIMENSIONS

 STAGNATION POINT
 DOWNGRADIENT(FT)
      -101.8591636
MAXIMUM UPGRADIENT
CAP.ZONE WIDTH(FT)
      -640.0000001

<<<<<<>>>>>>



CALCULATION NO. 4
```

```
+--------------------+
| REG.VEL.  INPUTS   |
+--------------------+

HYD.COND(GPD/FT2)=
              5000
  (or in FT/DAY)=
       668.4491979
  (or in CM/DAY)=
       20374.33155
HYD.GRAD(FT/FT)=
            0.0013
EFF.POROSITY(DEC)=
               0.3

REG.VELOCITY
  (FT/DAY)

  2.896613191

  (CM/DAY)

  88.28877006

<<<<<<>>>>>>



+--------------------+
| CAPT.ZONE  INPUTS  |
+--------------------+

REG.VEL.(FT/DAY)=
       2.896613191
EFF.SAT.THICK(FT)=
               100
EFF.POROSITY(DEC)=
               0.3
FLOW RATE(GPM)=
               100

CAPTURE  ZONE
  DIMENSIONS

 STAGNATION POINT
 DOWNGRADIENT(FT)
       35.25894124
```

```
MAXIMUM UPGRADIENT
CAP.ZONE WIDTH(FT)
       221.5384615

<<<<<<>>>>>>



CALCULATION NO. 5

+--------------------+
| CAPT.ZONE INPUTS   |
+--------------------+

REG.VEL.(FT/DAY)=
       2.896613191
EFF.SAT.THICK(FT)=
               100
EFF.POROSITY(DEC)=
               0.3
FLOW RATE(GPM)=
               200
CAPTURE ZONE
  DIMENSIONS
 STAGNATION POINT
 DOWNGRADIENT(FT)
       70.51788247
MAXIMUM UPGRADIENT
CAP.ZONE WIDTH(FT)
        443.076923

<<<<<<>>>>>>



CALCULATION NO. 6
```

```
+--------------------+
| CAPT.ZONE INPUTS   |
+--------------------+

REG.VEL.(FT/DAY)=
       2.896613191
EFF.SAT.THICK(FT)=
               100
EFF.POROSITY(DEC)=
               0.3
FLOW RATE(GPM)=
               500
CAPTURE ZONE
  DIMENSIONS
 STAGNATION POINT
 DOWNGRADIENT(FT)
       176.2947062
MAXIMUM UPGRADIENT
CAP.ZONE WIDTH(FT)
       1107.692308

<<<<<<>>>>>>



CALCULATION NO. 7

+--------------------+
| CAPT.ZONE INPUTS   |
+--------------------+

REG.VEL.(FT/DAY)=
       2.896613191
EFF.SAT.THICK(FT)=
                50
EFF.POROSITY(DEC)=
               0.3
FLOW RATE(GPM)=
               500
```

```
  CAPTURE ZONE
   DIMENSIONS
 STAGNATION POINT
 DOWNGRADIENT(FT)
       352.5894124
MAXIMUM UPGRADIENT
CAP.ZONE WIDTH(FT)
       2215.384615

<<<<<<>>>>>>



CALCULATION NO. 8

-----------------
|CAPT.ZONE INPUTS|
-----------------

REG.VEL.(FT/DAY)=
       2.896613191
EFF.SAT.THICK(FT)=
               200
EFF.POROSITY(DEC)=
               0.3
FLOW RATE(GPM)=
               500
  CAPTURE ZONE
   DIMENSIONS
 STAGNATION POINT
 DOWNGRADIENT(FT)
       88.14735309
MAXIMUM UPGRADIENT
CAP.ZONE WIDTH(FT)
       553.8461538

<<<<<<>>>>>>
```

```
CALCULATION NO. 9

-----------------
|CAPT.ZONE INPUTS|
-----------------

REG.VEL.(FT/DAY)=
       2.896613191
EFF.SAT.THICK(FT)=
               100
EFF.POROSITY(DEC)=
               0.2
FLOW RATE(GPM)=
               500
  CAPTURE ZONE
   DIMENSIONS
 STAGNATION POINT
 DOWNGRADIENT(FT)
       264.4420593
MAXIMUM UPGRADIENT
CAP.ZONE WIDTH(FT)
       1661.538462

<<<<<<>>>>>>



CALCULATION NO. 10

-----------------
| REG.VEL. INPUTS |
-----------------

HYD.COND(GPD/FT2)=
              5000
  (or in FT/DAY)=
       668.4491979
  (or in CM/DAY)=
       20374.33155
HYD.GRAD(FT/FT)=
            0.0013
```

```
EFF.POROSITY(DEC)=
                0.2
REG.VELOCITY
   (FT/DAY)
   4.344919786
   (CM/DAY)
   132.4331551
<<<<<<>>>>>>


+-------------------+
| CAPT.ZONE INPUTS  |
+-------------------+
REG.VEL.(FT/DAY)=
       4.344919786
EFF.SAT.THICK(FT)=
               100
EFF.POROSITY(DEC)=
               0.2
FLOW RATE(GPM)=
               500
CAPTURE ZONE
  DIMENSIONS
 STAGNATION POINT
 DOWNGRADIENT(FT)
       176.2947062
MAXIMUM UPGRADIENT
CAP.ZONE WIDTH(FT)
       1107.692308
<<<<<<>>>>>>
```

```
  PROGRAM
   OVER,
THANK YOU
```

Figure C.3. CAPZONE/VELDSTR Program Simulation Example No.2

```
CAPZONE
VELDSTR
PROGRAM

by J.F.KEELY


TIME= 62116.1112


CALCULATION NO. 1


-------------------------
   REG.VEL. INPUTS
-------------------------

HYD.COND(GPD/FT2)=
               750
  (or in FT/DAY)=
       100.2673797
  (or in CM/DAY)=
       3056.149733
HYD.GRAD(FT/FT)=
            0.0025
EFF.POROSITY(DEC)=
              0.25

REG.VELOCITY
   (FT/DAY)
   1.002673797
   (CM/DAY)
   30.56149733

<<<<<<>>>>>>
```

```
-------------------------
  VEL.DSTR. INPUTS
-------------------------

REG.VEL.(FT/DAY)=
       1.002673797
REG.FLOW DIRECTION
(COMPASS DEGREES)=
                 0
EFF.SAT.THICK(FT)=
               100
EFF.POROSITY(DEC)=
              0.25


 WELL NO. 1
FLOW RATE(GPM)=
                50
WELL X-COORD.(FT)=
               -50
WELL Y-COORD.(FT)=
                 0

 WELL NO. 2
FLOW RATE(GPM)=
                50
WELL X-COORD.(FT)=
                50
WELL Y-COORD.(FT)=
                 0



OBS.POINT 1
X-COORD.(FT)= 0
Y-COORD.(FT)=-40

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
                 0
VEL. Y-COMPONENT
(FT/DAY)=
       2.198359818
NET VEL.(FT/DAY)=
       2.198359818
```

```
NET FLOW DIRECTION
(COMPASS DEGREES)=
                 0

<<<<<<>>>>>>

OBS.POINT 2
X-COORD.(FT)= 0
Y-COORD.(FT)=-20

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
                 0
VEL. Y-COMPONENT
(FT/DAY)=
       1.847900122
NET VEL.(FT/DAY)=
       1.847900122
NET FLOW DIRECTION
(COMPASS DEGREES)=
                 0

<<<<<<>>>>>>

OBS.POINT 3
X-COORD.(FT)= 0
Y-COORD.(FT)= 0

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
                 0
VEL. Y-COMPONENT
(FT/DAY)=
       1.002673797
NET VEL.(FT/DAY)=
       1.002673797
NET FLOW DIRECTION
(COMPASS DEGREES)=
                 0

<<<<<<>>>>>>
```

```
OBS.POINT 4
X-COORD.(FT)= 0
Y-COORD.(FT)= 20

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
                 0
VEL. Y-COMPONENT
(FT/DAY)=
   1.574474718E-01
NET VEL.(FT/DAY)=
   1.574474718E-01
NET FLOW DIRECTION
(COMPASS DEGREES)=
                 0

<<<<<<>>>>>>

OBS.POINT 5
X-COORD.(FT)= 0
Y-COORD.(FT)= 40

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
                 0
VEL. Y-COMPONENT
(FT/DAY)=
      -0.193012224
NET VEL.(FT/DAY)=
       0.193012224
NET FLOW DIRECTION
(COMPASS DEGREES)=
               180

<<<<<<>>>>>>

OBS.POINT 6
X-COORD.(FT)= 20
Y-COORD.(FT)=-40

<<<<<<>>>>>>
```

```
VEL. X-COMPONENT
(FT/DAY)=
    7.54201952E-02
VEL. Y-COMPONENT
(FT/DAY)=
        2.36023731
NET VEL.(FT/DAY)=
        2.36144201
NET FLOW DIRECTION
(COMPASS DEGREES)=
       1.830235019

<<<<<<>>>>>>

OBS.POINT 7

X-COORD.(FT)= 20
Y-COORD.(FT)=-20

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
   6.047845835E-01
VEL. Y-COMPONENT
(FT/DAY)=
       2.176667401
NET VEL.(FT/DAY)=
       2.259124912
NET FLOW DIRECTION
(COMPASS DEGREES)=
       15.52789257

<<<<<<>>>>>>

OBS.POINT 8

X-COORD.(FT)= 20
Y-COORD.(FT)= 0

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
       1.167217306
VEL. Y-COMPONENT
(FT/DAY)=
       1.002673797
```

```
NET VEL.(FT/DAY)=
       1.538749812
NET FLOW DIRECTION
(COMPASS DEGREES)=
       49.33645739

<<<<<<>>>>>>

OBS.POINT 9

X-COORD.(FT)= 20
Y-COORD.(FT)= 20

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
   6.047845835E-01
VEL. Y-COMPONENT
(FT/DAY)=
      -0.171319807
NET VEL.(FT/DAY)=
   6.285816325E-01
NET FLOW DIRECTION
(COMPASS DEGREES)=
       105.8160521

<<<<<<>>>>>>

OBS.POINT 10

X-COORD.(FT)= 20
Y-COORD.(FT)= 40

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
    7.54201952E-02
VEL. Y-COMPONENT
(FT/DAY)=
      -0.354889716
NET VEL.(FT/DAY)=
   3.628152648E-01
NET FLOW DIRECTION
(COMPASS DEGREES)=
       168.0021537
```

```
<<<<<<>>>>>>

OBS.POINT 11
X-COORD.(FT)= 40
Y-COORD.(FT)=-40

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
  -2.081030248E-01
VEL. Y-COMPONENT
(FT/DAY)=
       2.697226999
NET VEL.(FT/DAY)=
       2.705243123
NET FLOW DIRECTION
(COMPASS DEGREES)=
        355.588117

<<<<<<>>>>>>

OBS.POINT 12
X-COORD.(FT)= 40
Y-COORD.(FT)=-20

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
   5.767426685E-01
VEL. Y-COMPONENT
(FT/DAY)=
       3.598015807
NET VEL.(FT/DAY)=
       3.643947016
NET FLOW DIRECTION
(COMPASS DEGREES)=
       9.106736721

<<<<<<>>>>>>

OBS.POINT 13
X-COORD.(FT)= 40
Y-COORD.(FT)= 0

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
       5.447014095
VEL. Y-COMPONENT
(FT/DAY)=
       1.002673797
NET VEL.(FT/DAY)=
       5.538530247
NET FLOW DIRECTION
(COMPASS DEGREES)=
       79.56988574

<<<<<<>>>>>>

OBS.POINT 14
X-COORD.(FT)= 40
Y-COORD.(FT)= 20

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
   5.767426685E-01
VEL. Y-COMPONENT
(FT/DAY)=
      -1.592668213
NET VEL.(FT/DAY)=
       1.693878432
NET FLOW DIRECTION
(COMPASS DEGREES)=
       160.0934855

<<<<<<>>>>>>

OBS.POINT 15
X-COORD.(FT)= 40
Y-COORD.(FT)= 40

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
  -2.081030248E-01
VEL. Y-COMPONENT
(FT/DAY)=
```

```
       -0.691879405
NET VEL.(FT/DAY)=
   7.224984291E-01
NET FLOW DIRECTION
(COMPASS DEGREES)=
        196.7402009

<<<<<<>>>>>>

OBS.POINT 16

X-COORD.(FT)= 60
Y-COORD.(FT)=-40

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
  -8.524846019E-01
VEL. Y-COMPONENT
(FT/DAY)=
         2.62344699
NET VEL.(FT/DAY)=
        2.758478585
NET FLOW DIRECTION
(COMPASS DEGREES)=
        341.9985184

<<<<<<>>>>>>

OBS.POINT 17

X-COORD.(FT)= 60
Y-COORD.(FT)=-20

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
       -1.764832567
VEL. Y-COMPONENT
(FT/DAY)=
        3.551876394
NET VEL.(FT/DAY)=
        3.966164382
NET FLOW DIRECTION
(COMPASS DEGREES)=
        333.5784451
```

```
<<<<<<>>>>>>

OBS.POINT 18

X-COORD.(FT)= 60
Y-COORD.(FT)= 0

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
       -6.684971844
VEL. Y-COMPONENT
(FT/DAY)=
        1.002673797
NET VEL.(FT/DAY)=
         6.75974876
NET FLOW DIRECTION
(COMPASS DEGREES)=
        278.5301623

<<<<<<>>>>>>

OBS.POINT 19

X-COORD.(FT)= 60
Y-COORD.(FT)= 20

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
       -1.764832567
VEL. Y-COMPONENT
(FT/DAY)=
         -1.5465288
NET VEL.(FT/DAY)=
         2.34656884
NET FLOW DIRECTION
(COMPASS DEGREES)=
        228.7718055

<<<<<<>>>>>>

OBS.POINT 20

X-COORD.(FT)= 60
Y-COORD.(FT)= 40
```

```
<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
  -8.524846019E-01
VEL. Y-COMPONENT
(FT/DAY)=
       -0.618099396
NET VEL.(FT/DAY)=
        1.052984739
NET FLOW DIRECTION
(COMPASS DEGREES)=
         234.055771

<<<<<<>>>>>>

OBS.POINT 21

X-COORD.(FT)= 80
Y-COORD.(FT)=-40

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
        -1.16595545
VEL. Y-COMPONENT
(FT/DAY)=
        2.115631272
NET VEL.(FT/DAY)=
        2.415646454
NET FLOW DIRECTION
(COMPASS DEGREES)=
        331.1401956
<<<<<<>>>>>>

OBS.POINT 22

X-COORD.(FT)= 80
Y-COORD.(FT)=-20

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
       -1.874606007
VEL. Y-COMPONENT
(FT/DAY)=
```

```
        2.016268905
NET VEL.(FT/DAY)=
        2.753086991
NET FLOW DIRECTION
(COMPASS DEGREES)=
        317.0851615

<<<<<<>>>>>>

OBS.POINT 23

X-COORD.(FT)= 80
Y-COORD.(FT)= 0

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
       -2.514006506
VEL. Y-COMPONENT
(FT/DAY)=
        1.002673797
NET VEL.(FT/DAY)=
        2.706581507
NET FLOW DIRECTION
(COMPASS DEGREES)=
        291.7438439

<<<<<<>>>>>>

OBS.POINT 24

X-COORD.(FT)= 80
Y-COORD.(FT)= 20
<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
       -1.874606007
VEL. Y-COMPONENT
(FT/DAY)=
       -0.010921311
NET VEL.(FT/DAY)=
         1.87463782
NET FLOW DIRECTION
(COMPASS DEGREES)=
         269.666203
```

```
<<<<<<>>>>>>

OBS.POINT 25
X-COORD.(FT)= 80
Y-COORD.(FT)= 40

<<<<<<>>>>>>
VEL. X-COMPONENT
(FT/DAY)=
       -1.16595545
VEL. Y-COMPONENT
(FT/DAY)=
      -0.110283678
NET VEL.(FT/DAY)=
       1.171159511
NET FLOW DIRECTION
(COMPASS DEGREES)=
       264.5966667

<<<<<<>>>>>>

OBS.POINT 26
X-COORD.(FT)= 100
Y-COORD.(FT)=-40
<<<<<<>>>>>>
VEL. X-COMPONENT
(FT/DAY)=
      -1.128707758
VEL. Y-COMPONENT
(FT/DAY)=
        1.70222454
NET VEL.(FT/DAY)=
       2.042437169
NET FLOW DIRECTION
(COMPASS DEGREES)=
       326.4525148

<<<<<<>>>>>>

OBS.POINT 27
X-COORD.(FT)= 100
Y-COORD.(FT)=-20
<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
      -1.457923137
VEL. Y-COMPONENT
(FT/DAY)=
       1.478805657
NET VEL.(FT/DAY)=
       2.076633344
NET FLOW DIRECTION
(COMPASS DEGREES)=
       315.4074125

<<<<<<>>>>>>

OBS.POINT 28
X-COORD.(FT)= 100
Y-COORD.(FT)= 0

<<<<<<>>>>>>
VEL. X-COMPONENT
(FT/DAY)=
      -1.634104228
VEL. Y-COMPONENT
(FT/DAY)=
       1.002673797
NET VEL.(FT/DAY)=
       1.917198835
NET FLOW DIRECTION
(COMPASS DEGREES)=
       301.5329585

<<<<<<>>>>>>

OBS.POINT 29
X-COORD.(FT)= 100
Y-COORD.(FT)= 20
<<<<<<>>>>>>
VEL. X-COMPONENT
(FT/DAY)=
      -1.457923137
VEL. Y-COMPONENT
(-T/DAY)=
       0.526541937
NET VEL.(FT/DAY)=
       1.550092347
NET FLOW DIRECTION
```

```
(COMPASS DEGREES)=
       289.8576373
<<<<<<>>>>>>
OBS.POINT 30

X-COORD.(FT)= 100
Y-COORD.(FT)= 40

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
      -1.128707758
(EL./DAY-COMPONENT
   3.031230544E-01
NET VEL.(FT/DAY)=
       1.168702182
NET FLOW DIRECTION
(COMPASS DEGREES)=
       285.0325195

<<<<<<>>>>>>
  PROGRAM
   OVER,
THANK YOU
```

Figure C.4. CAPZONE/VELDSTR Program Simulation Example No.3

```
CAPZONE
VELDSTR
PROGRAM

by J.F.KEELY


TIME= 62519.0233


CALCULATION NO. 1


VEL.DSTR. INPUTS

REG.VEL.(FT/DAY)=
                 0
REG.FLOW DIRECTION
(COMPASS DEGREES)=
                90
EFF.SAT.THICK(FT)=
               100
EFF.POROSITY(DEC)=
               0.3

 WELL NO. 1
FLOW RATE(GPM)=
               500
WELL X-COORD.(FT)=
                 0
WELL Y-COORD.(FT)=
                 0


OBS.POINT 1
X-COORD.(FT)=-500
Y-COORD.(FT)= 0
<<<<<<>>>>>>
VEL. X-COMPONENT
(FT/DAY)=
      1.021315143
VEL. Y-COMPONENT
(FT/DAY)=
                0
NET VEL.(FT/DAY)=
      1.021315143
NET FLOW DIRECTION
(COMPASS DEGREES)=
               90
<<<<<<>>>>>>

OBS.POINT 2
X-COORD.(FT)=-400
Y-COORD.(FT)= 0
<<<<<<>>>>>>
VEL. X-COMPONENT
(FT/DAY)=
      1.276643929
VEL. Y-COMPONENT
(FT/DAY)=
                0
NET VEL.(FT/DAY)=
      1.276643929
NET FLOW DIRECTION
(COMPASS DEGREES)=
               90
<<<<<<>>>>>>

OBS.POINT 3
X-COORD.(FT)=-300
Y-COORD.(FT)= 0
<<<<<<>>>>>>
VEL. X-COMPONENT
(FT/DAY)=
      1.702191905
VEL. Y-COMPONENT
(FT/DAY)=
```

```
                  0
NET VEL.(FT/DAY)=
      1.702191905
NET FLOW DIRECTION
(COMPASS DEGREES)=
                 90

<<<<<<>>>>>>

OBS.POINT 4

X-COORD.(FT)=-200
Y-COORD.(FT)= 0

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
      2.553287857
VEL. Y-COMPONENT
(FT/DAY)=
                  0
NET VEL.(FT/DAY)=
      2.553287857
NET FLOW DIRECTION
(COMPASS DEGREES)=
                 90

<<<<<<>>>>>>

OBS.POINT 5

X-COORD.(FT)=-100
Y-COORD.(FT)= 0

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
      5.106575714
VEL. Y-COMPONENT
(FT/DAY)=
                  0
NET VEL.(FT/DAY)=
      5.106575714
NET FLOW DIRECTION
(COMPASS DEGREES)=
                 90
```

```
<<<<<<>>>>>>

OBS.POINT 6

X-COORD.(FT)= 100
Y-COORD.(FT)= 0

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
     -5.106575714
VEL. Y-COMPONENT
(FT/DAY)=
                  0
NET VEL.(FT/DAY)=
      5.106575714
NET FLOW DIRECTION
(COMPASS DEGREES)=
                270

<<<<<<>>>>>>

OBS.POINT 7

X-COORD.(FT)= 200
Y-COORD.(FT)= 0

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
     -2.553287857
VEL. Y-COMPONENT
(FT/DAY)=
                  0
NET VEL.(FT/DAY)=
      2.553287857
NET FLOW DIRECTION
(COMPASS DEGREES)=
                270

<<<<<<>>>>>>

OBS.POINT 8

X-COORD.(FT)= 300
Y-COORD.(FT)= 0
```

```
<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
      -1.702191905
VEL. Y-COMPONENT
(FT/DAY)=
                 0
NET VEL.(FT/DAY)=
       1.702191905
NET FLOW DIRECTION
(COMPASS DEGREES)=
               270

<<<<<<>>>>>>
OBS.POINT 9

X-COORD.(FT)= 400
Y-COORD.(FT)= 0

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
      -1.276643929
VEL. Y-COMPONENT
(FT/DAY)=
                 0
NET VEL.(FT/DAY)=
       1.276643929
NET FLOW DIRECTION
(COMPASS DEGREES)=
               270

<<<<<<>>>>>>

OBS.POINT 10

X-COORD.(FT)= 500
Y-COORD.(FT)= 0

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
      -1.021315143
VEL. Y-COMPONENT
(FT/DAY)=
                 0
```

```
NET VEL.(FT/DAY)=
       1.021315143
NET FLOW DIRECTION
(COMPASS DEGREES)=
               270

<<<<<<>>>>>>

OBS.POINT 11

X-COORD.(FT)= 0
Y-COORD.(FT)=-500

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
                 0
VEL. Y-COMPONENT
(FT/DAY)=
       1.021315143
NET VEL.(FT/DAY)=
       1.021315143
NET FLOW DIRECTION
(COMPASS DEGREES)=
                 0
<<<<<<>>>>>>

OBS.POINT 12

X-COORD.(FT)= 0
Y-COORD.(FT)=-400

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
                 0
VEL. Y-COMPONENT
(FT/DAY)=
       1.276643929
NET VEL.(FT/DAY)=
       1.276643929
NET FLOW DIRECTION
(COMPASS DEGREES)=
                 0
<<<<<<>>>>>>
```

```
OBS.POINT 13
X-COORD.(FT)= 0
Y-COORD.(FT)=-300

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
                   0
VEL. Y-COMPONENT
(FT/DAY)=
         1.702191905
NET VEL.(FT/DAY)=
         1.702191905
NET FLOW DIRECTION
(COMPASS DEGREES)=
                   0

<<<<<<>>>>>>

OBS.POINT 14
X-COORD.(FT)= 0
Y-COORD.(FT)=-200

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
                   0
VEL. Y-COMPONENT
(FT/DAY)=
         2.553287857
NET VEL.(FT/DAY)=
         2.553287857
NET FLOW DIRECTION
(COMPASS DEGREES)=
                   0

<<<<<<>>>>>>

OBS.POINT 15
X-COORD.(FT)= 0
Y-COORD.(FT)=-100
<<<<<<>>>>>>
VEL. X-COMPONENT
(FT/DAY)=
                   0
VEL. Y-COMPONENT
(FT/DAY)=
         5.106575714
NET VEL.(FT/DAY)=
         5.106575714
NET FLOW DIRECTION
(COMPASS DEGREES)=
                   0

<<<<<<>>>>>>

OBS.POINT 16
X-COORD.(FT)= 0
Y-COORD.(FT)= 100

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
                   0
VEL. Y-COMPONENT
(FT/DAY)=
        -5.106575714
NET VEL.(FT/DAY)=
         5.106575714
NET FLOW DIRECTION
(COMPASS DEGREES)=
                 180

<<<<<<>>>>>>

OBS.POINT 17
X-COORD.(FT)= 0

Y-COORD.(FT)= 200
<<<<<<>>>>>>
VEL. X-COMPONENT
(FT/DAY)=
                   0
VEL. Y-COMPONENT
(FT/DAY)=
        -2.553287857
NET VEL.(FT/DAY)=
         2.553287857
NET FLOW DIRECTION
(COMPASS DEGREES)=
                 180
```

```
<<<<<<>>>>>>

OBS.POINT 18
X-COORD.(FT)= 0
Y-COORD.(FT)= 300

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
                0
VEL. Y-COMPONENT
(FT/DAY)=
     -1.702191905
NET VEL.(FT/DAY)=
      1.702191905
NET FLOW DIRECTION
(COMPASS DEGREES)=
              180

<<<<<<>>>>>>

OBS.POINT 19
X-COORD.(FT)= 0
Y-COORD.(FT)= 400

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
                0
VEL. Y-COMPONENT
(FT/DAY)=
     -1.276643929
NET VEL.(FT/DAY)=
      1.276643929
NET FLOW DIRECTION
(COMPASS DEGREES)=
              180

<<<<<<>>>>>>

OBS.POINT 20
X-COORD.(FT)= 0
Y-COORD.(FT)= 500
<<<<<<>>>>>>
```

```
VEL. X-COMPONENT
(FT/DAY)=
                0
VEL. Y-COMPONENT
(FT/DAY)=
     -1.021315143
NET VEL.(FT/DAY)=
      1.021315143
NET FLOW DIRECTION
(COMPASS DEGREES)=
              180

<<<<<<>>>>>>

  PROGRAM
   OVER,
THANK YOU
```

Figure C.5. CAPZONE/VELDSTR Program Simulation Example No.4

```
CAPZONE
VELDSTR
PROGRAM

by J.F.KEELY


TıME= 62617.3631



CALCULATION NO. 1


┌───────────────────┐
│ VEL.DSTR. INPUTS  │
└───────────────────┘

REG.VEL.(FT/DAY)=
      2.896613191
REG.FLOW DIRECTION
(COMPASS DEGREES)=
               90
EFF.SAT.THICK(FT)=
              100
EFF.POROSITY(DEC)=
              0.3


  WELL NO. 1
FLOW RATE(GPM)=
              500
WELL X-COORD.(FT)=
                0
WELL Y-COORD.(FT)=
                0



OBS.POINT 1
X-COORD.(FT)=-500
Y-COORD.(FT)= 0
```

```
<<<<<<>>>>>>
VEL. X-COMPONENT
(FT/DAY)=
      3.917928334
VEL. Y-COMPONENT
(FT/DAY)=
                0
NET VEL.(FT/DAY)=
      3.917928334
NET FLOW DIRECTION
(COMPASS DEGREES)=
               90

<<<<<<>>>>>>

OBS.POINT 2
X-COORD.(FT)=-400
Y-COORD.(FT)= 0

<<<<<<>>>>>>
VEL. X-COMPONENT
(FT/DAY)=
       4.17325712
VEL. Y-COMPONENT
(FT/DAY)=
                0
NET VEL.(FT/DAY)=
       4.17325712
NET FLOW DIRECTION
(COMPASS DEGREES)=
               90

<<<<<<>>>>>>

OBS.POINT 3
X-COORD.(FT)=-300
Y-COORD.(FT)= 0

<<<<<<>>>>>>
VEL. X-COMPONENT
(FT/DAY)=
      4.598805096
VEL. Y-COMPONENT
(FT/DAY)=
```

```
                 0
NET VEL.(FT/DAY)=
      4.598805096
NET FLOW DIRECTION
(COMPASS DEGREES)=
               90

<<<<<<>>>>>>

OBS.POINT 4
X-COORD.(FT)=-200
Y-COORD.(FT)= 0

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
      5.449901048
VEL. Y-COMPONENT
(FT/DAY)=
                0
NET VEL.(FT/DAY)=
      5.449901048
NET FLOW DIRECTION
(COMPASS DEGREES)=
               90

<<<<<<>>>>>>

OBS.POINT 5
X-COORD.(FT)=-100
Y-COORD.(FT)= 0

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
      8.003188905
VEL. Y-COMPONENT
(FT/DAY)=
                0
NET VEL.(FT/DAY)=
      8.003188905
NET FLOW DIRECTION
(COMPASS DEGREES)=
               90
```

```
<<<<<<>>>>>>

OBS.POINT 6
X-COORD.(FT)= 100
Y-COORD.(FT)= 0

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
     -2.209962523
VEL. Y-COMPONENT
(FT/DAY)=
                0
NET VEL.(FT/DAY)=
      2.209962523
NET FLOW DIRECTION
(COMPASS DEGREES)=
              270

<<<<<<>>>>>>

OBS.POINT 7
X-COORD.(FT)= 200
Y-COORD.(FT)= 0

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
      0.343325334
VEL. Y-COMPONENT
(FT/DAY)=
                0
NET VEL.(FT/DAY)=
      0.343325334
NET FLOW DIRECTION
(COMPASS DEGREES)=
               90

<<<<<<>>>>>>

OBS.POINT 8
X-COORD.(FT)= 300
Y-COORD.(FT)= 0
```

```
<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
       1.194421286
VEL. Y-COMPONENT
(FT/DAY)=
                 0
NET VEL.(FT/DAY)=
       1.194421286
NET FLOW DIRECTION
(COMPASS DEGREES)=
                90

<<<<<<>>>>>>

OBS.POINT 9

X-COORD.(FT)= 400
Y-COORD.(FT)= 0

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
       1.619969262
VEL. Y-COMPONENT
(FT/DAY)=
                 0
NET VEL.(FT/DAY)=
       1.619969262
NET FLOW DIRECTION
(COMPASS DEGREES)=
                90

<<<<<<>>>>>>

OBS.POINT 10

X-COORD.(FT)= 500
Y-COORD.(FT)= 0

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
       1.875298048
VEL. Y-COMPONENT
(FT/DAY)=
```

```
                 0
NET VEL.(FT/DAY)=
       1.875298048
NET FLOW DIRECTION
(COMPASS DEGREES)=
                90

<<<<<<>>>>>>

OBS.POINT 11

X-COORD.(FT)= 0
Y-COORD.(FT)=-500

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
       2.896613191
VEL. Y-COMPONENT
(FT/DAY)=
       1.021315143
NET VEL.(FT/DAY)=
       3.071392616
NET FLOW DIRECTION
(COMPASS DEGREES)=
       70.57788924

<<<<<<>>>>>>

OBS.POINT 12

X-COORD.(FT)= 0
Y-COORD.(FT)=-400

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
       2.896613191
VEL. Y-COMPONENT
(FT/DAY)=
       1.276643929
NET VEL.(FT/DAY)=
       3.165468006
NET FLOW DIRECTION
(_OMPASS DEGREES)=
       66.21514853
<<<<<<>>>>>>
```

```
OBS.POINT 13
X-COORD.(FT)= 0
Y-COORD.(FT)=-300
<<<<<<>>>>>>
VEL. X-COMPONENT
(FT/DAY)=
       2.896613191
VEL. Y-COMPONENT
(FT/DAY)=
       1.702191905
NET VEL.(FT/DAY)=
       3.359735891
NET FLOW DIRECTION
(COMPASS DEGREES)=
       59.55941725
<<<<<<>>>>>>
OBS.POINT 14
X-COORD.(FT)= 0
Y-COORD.(FT)=-200
<<<<<<>>>>>>
VEL. X-COMPONENT
(FT/DAY)=
       2.896613191
VEL. Y-COMPONENT
(FT/DAY)=
       2.553287857
NET VEL.(FT/DAY)=
       3.861301187
NET FLOW DIRECTION
(COMPASS DEGREES)=
       48.60467689
<<<<<<>>>>>>
OBS.POINT 15
X-COORD.(FT)= 0
Y-COORD.(FT)=-100
<<<<<<>>>>>>
```

```
VEL. X-COMPONENT
(FT/DAY)=
       2.896613191
VEL. Y-COMPONENT
(FT/DAY)=
       5.106575714
NET VEL.(FT/DAY)=
       5.870901421
NET FLOW DIRECTION
(COMPASS DEGREES)=
       29.56329478
<<<<<<>>>>>>
OBS.POINT 16
X-COORD.(FT)= 0
Y-COORD.(FT)= 100
<<<<<<>>>>>>
VEL. X-COMPONENT
(FT/DAY)=
       2.896613191
VEL. Y-COMPONENT
(FT/DAY)=
      -5.106575714
NET VEL.(FT/DAY)=
       5.870901421
NET FLOW DIRECTION
(COMPASS DEGREES)=
       150.4367052
<<<<<<>>>>>>
OBS.POINT 17
X-COORD.(FT)= 0
Y-COORD.(FT)= 200
<<<<<<>>>>>>
VEL. X-COMPONENT
(FT/DAY)=
       2.896613191
VEL. Y-COMPONENT
(FT/DAY)=
      -2.553287857
```

```
NET VEL.(FT/DAY)=
        3.861301187
NET FLOW DIRECTION
(COMPASS DEGREES)=
        131.3953231

<<<<<<>>>>>>

OBS.POINT 18
X-COORD.(FT)= 0
Y-COORD.(FT)= 300

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
        2.896613191
VEL. Y-COMPONENT
(FT/DAY)=
       -1.702191905
NET VEL.(FT/DAY)=
        3.359735891
NET FLOW DIRECTION
(COMPASS DEGREES)=
        120.4405828

<<<<<<>>>>>>

OBS.POINT 19
X-COORD.(FT)= 0
Y-COORD.(FT)= 400

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
        2.896613191
VEL. Y-COMPONENT
(FT/DAY)=
       -1.276643929
NET VEL.(FT/DAY)=
        3.165468006
NET FLOW DIRECTION
(COMPASS DEGREES)=
        113.7848515

<<<<<<>>>>>>
```

```
OBS.POINT 20
X-COORD.(FT)= 0
Y-COORD.(FT)= 500

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
        2.896613191
VEL. Y-COMPONENT
(FT/DAY)=
       -1.021315143
NET VEL.(FT/DAY)=
        3.071392616
NET FLOW DIRECTION
(COMPASS DEGREES)=
        109.4221108

<<<<<<>>>>>>

OBS.POINT 21
X-COORD.(FT)= 176.
2947062
Y-COORD.(FT)= 0

<<<<<<>>>>>>

VEL. X-COMPONENT
(FT/DAY)=
                  0
VEL. Y-COMPONENT
(FT/DAY)=
                  0
NET VEL.(FT/DAY)=
                  0
NET FLOW DIRECTION
(COMPASS DEGREES)=
                270

<<<<<<>>>>>>


  PROGRAM
   OVER,
THANK YOU
```

The results of additional well hydraulics simulations are given in Table C.1 and Figures C.6 through C.10. These simulations are typical of calculations required for production and injections wells in oilfield operations, and use oilfield units (e.g., intrinsic permeability in millidarcies and flow rates in barrels per day) as a result. They were performed using the program presented by Warner and Yow (1979), and their interpretation follows the same lines of reasoning as is presented in Chapter 3 for the simulations of pumping wells that used common ground water units. In other words, the net pressure changes at a given observation point is the arithmetic sum of the individual pressure changes of the wells involved.

The notes at the bottom of Table 1 list the data input used for these simulations.

Table C.1. Pressure Changes Induced by Pumpage or Injection of 1000 Barrels per Day in a Hypothetical Reservoir

	TIME (DAYS)						
RADIUS (FEET)	0.01	0.1	1	10	100	1000	10000
1	69.54	85.79	102.05	118.31	134.56	150.81	167.07
2	59.75	76.01	92.26	108.52	124.77	141.03	157.29
3	54.03	70.28	86.54	102.79	119.05	135.31	151.56
4	49.97	66.22	82.47	98.73	114.99	131.24	147.50
5	46.82	63.07	79.32	95.58	111.84	128.09	144.35
6	44.24	60.49	76.75	93.01	109.26	125.52	141.77
7	42.07	58.32	74.57	90.83	107.09	123.34	139.60
8	40.19	56.43	72.69	88.94	105.20	121.46	137.71
9	38.53	54.77	71.02	87.28	103.54	119.79	136.05
10	37.05	53.28	69.54	85.79	102.05	118.31	134.56
20	27.32	43.50	59.75	76.01	92.26	108.52	124.77
30	21.70	37.79	54.03	70.28	86.54	102.79	119.05
40	17.78	33.74	49.97	66.22	82.47	98.73	114.99
50	14.81	30.61	46.82	63.07	79.32	95.58	111.84
60	12.46	28.06	44.24	60.49	76.75	93.01	109.26
70	10.54	25.91	42.07	58.32	74.57	90.83	107.09
80	8.94	24.05	40.19	56.43	72.69	88.94	105.20
90	7.60	22.42	38.53	54.77	71.02	87.28	103.54
100	6.46	20.98	37.05	53.28	69.54	85.79	102.05
200	1.15	11.79	27.32	43.50	59.75	76.01	92.26
300	0.0	7.02	21.70	37.79	54.03	70.28	86.54
400	0.0	4.18	17.78	33.74	49.97	66.22	82.47
500	0.0	2.44	14.81	30.61	46.82	63.07	79.32
600	0.0	1.39	12.46	28.06	44.24	60.49	76.75
700	0.0	0.76	10.54	25.91	42.07	58.32	74.57
800	0.0	0.34	8.94	24.05	40.19	56.43	72.69
900	0.0	0.21	7.59	22.42	38.53	54.77	71.02
1000	0.0	0.10	6.46	20.98	37.05	53.28	69.54
2000	0.0	0.0	1.15	11.79	27.32	43.50	59.75
3000	0.0	0.0	0.14	7.02	21.70	37.79	54.03
4000	0.0	0.0	0.01	4.18	17.78	33.74	49.97
5000	0.0	0.0	0.0	2.44	14.81	30.61	46.82
6000	0.0	0.0	0.0	1.39	12.46	28.06	44.24
7000	0.0	0.0	0.0	0.76	10.54	25.91	42.07
8000	0.0	0.0	0.0	0.34	8.94	24.05	40.19
9000	0.0	0.0	0.0	0.21	7.60	22.42	38.53
10000	0.0	0.0	0.0	0.10	6.46	20.98	37.05

Notes: The intrinsic permeability of the reservoir is 100 millidarcies, its saturated thickness is 100 feet, and its effective porosity is 10%. It contains fluids that have a density equal to that of fresh water and a viscosity of 1.00 centipoise. The system compressibility is 7.5 x 10^{-6} psi^{-1}. The formation volume factor is 1.00 reservoir barrels/standard barrels, and the skin factor is assumed to be zero.

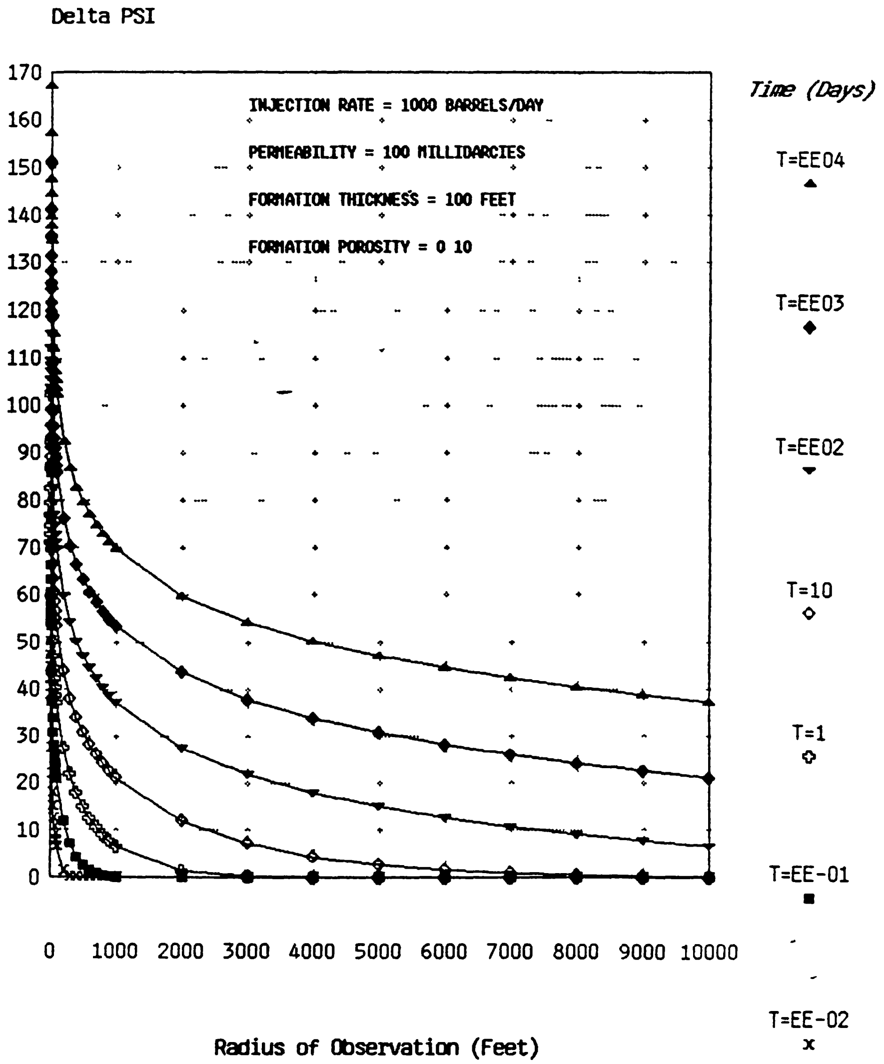


Figure C.6. Master Plot of Theis Solutions Given in Table C.1

Note: See Table C.1 notes for additional data.

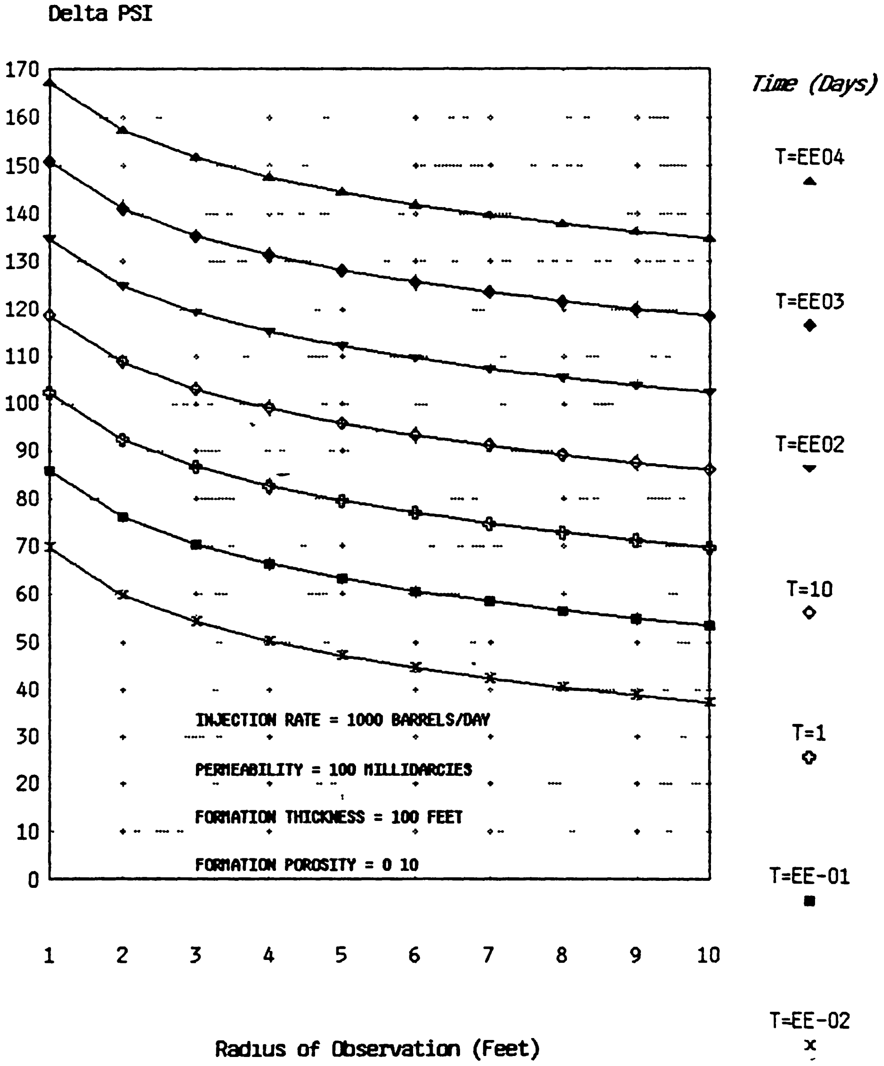


Figure C.7. High Resolution Plot (for 1 - 10 Foot Radii of Observation) of Theis Solutions Given in Table C.1

Note: See Table C.1 notes for additional data.

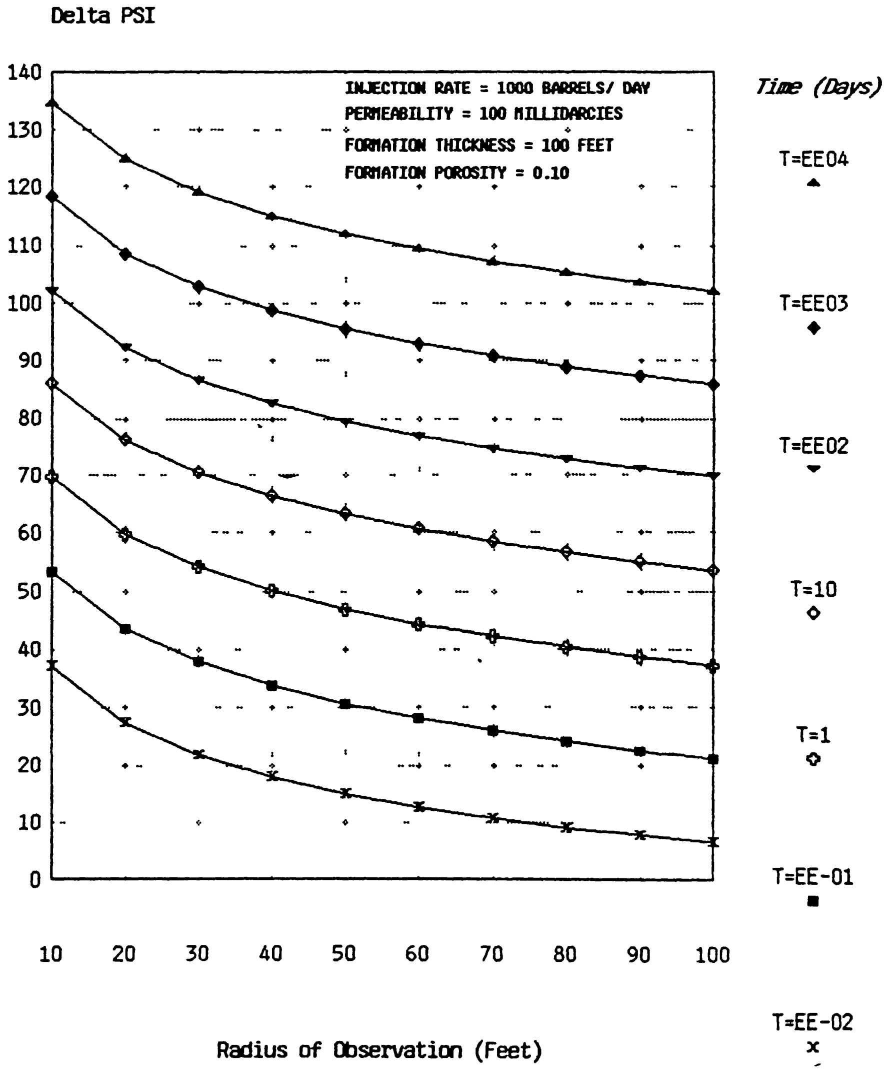


Figure C.8. High Resolution Plot (for 10 - 100 Foot Radii of Observation) of Theis Solutions Given in Table C.1

Note: See Table C.1 notes for additional data.

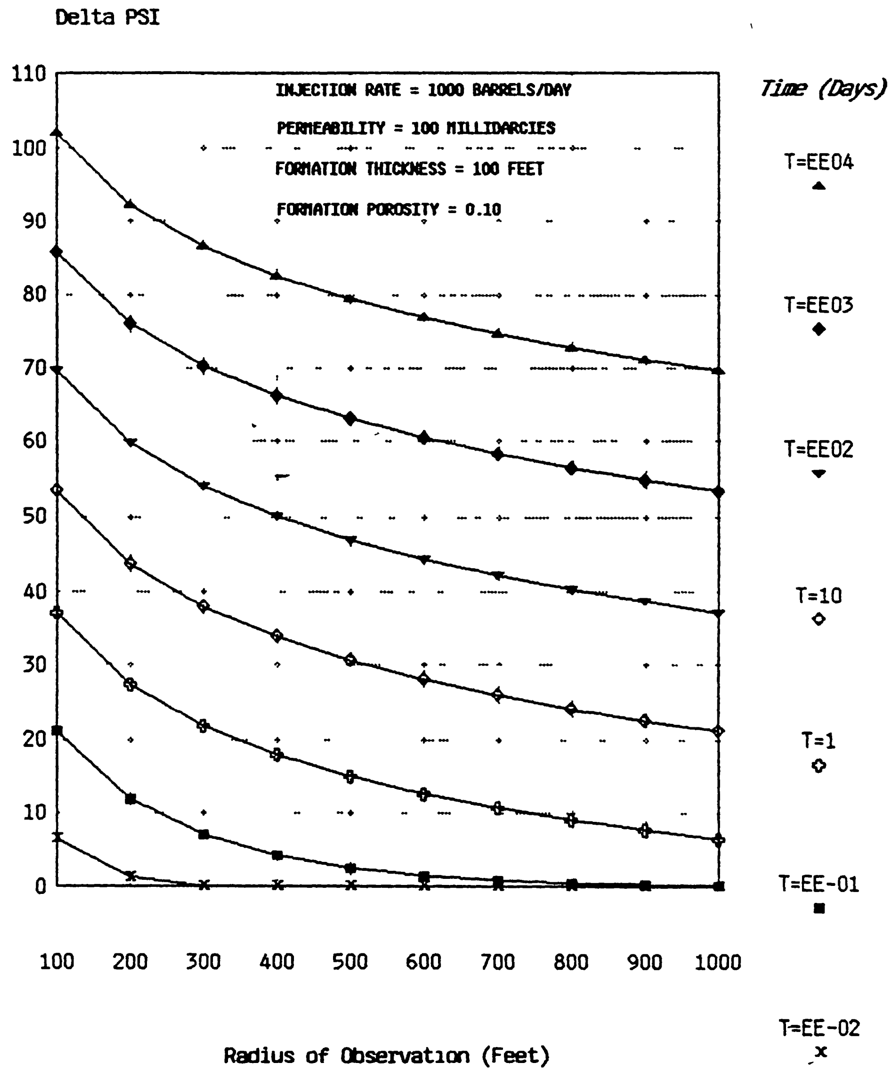


Figure C.9. High Resolution Plot (for 100 - 1,000 Foot Radii of Observation) of Theis Solutions Given in Table C.1

Note: See Table C.1 notes for additional data.

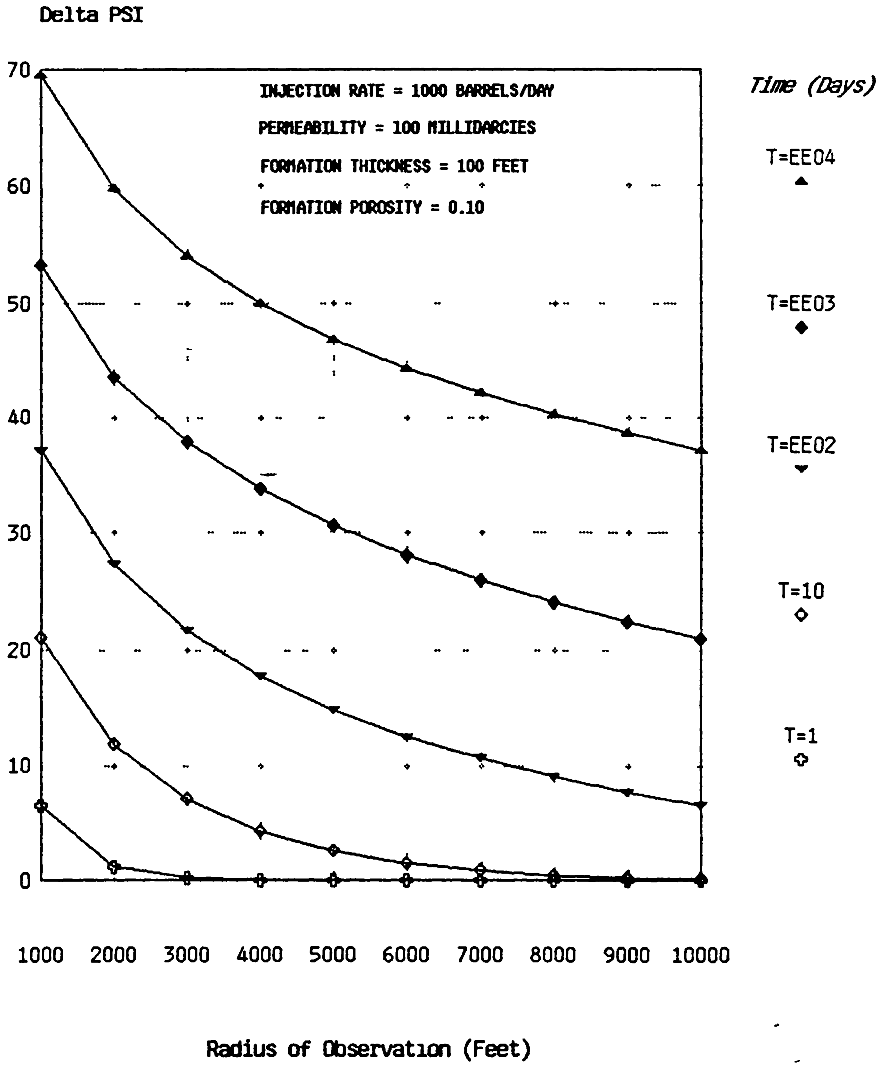


Figure C.10. High Resolution Plot (for 1,000 - 10,000 Foot Radii of Observation) of Theis Solutions Given in Table C.1

2

VITA

Joseph Francis Keely, Jr.

Candidate for the Degree of

Doctor of Philosophy

Thesis: GROUND-WATER CONTAMINATION ASSESSMENTS

Major Field: Civil Engineering

Biographical:

Personal Data: Born in Seattle, Washington, April 13,1954, the son of Joseph Francis Sr. and Margaret Ann Barton Keely. Married to Rosemary Church on June 15, 1974.

Education: Graduated from Boise High School, Boise, Idaho, in May, 1972; received Bachelor of Science in Professional Chemistry from University of Idaho in December, 1975; received Master of Science degree from University of Idaho in December, 1979; completed requirements for the Doctor of Philosophy degree at Oklahoma State University in July,1986.

Scientific Societies: Sigma Xi Scientific Research Society of North America, 1976; American Chemical Society, 1976; American Association for the Advancement of Science, 1977; Association of Ground-Water Scientists and Engineers (Ground-Water Technology Division of National Water Well Association), 1981; American Institute of Hydrology (Certified Professional Hydrogeologist no. 483), 1984.

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Professional Service: Invited Participant and Speaker, NATO Advanced Study Institute on Mechanics of Fluids in Porous Media, University of Delaware, Newark, Delaware, July, 1982; Member, Policy Board, International Ground Water Modeling Center, Indianapolis, Indiana and Delft, The Netherlands, 1982- ; Session Moderator, Speaker, and Panelist, National Symposium on Aquifer Restoration and Ground-Water Monitoring, Columbus, Ohio, 1983, 1984, and 1985; Member, Hydraulics Division Task Committee on Organics in Drinking Water, American Society of Civil Engineers, 1983-1985; Member, Aquifer Protection Committee, Association of Ground-Water Scientists and Engineers, NWWA, 1983- ; Member, Ground Water Editorial Board, Association of Ground-Water Scientists and Engineers, NWWA, 1984- , Invited Participant and Speaker, NATO Advanced Study Institute on Fundamentals of Transport Phenomena in Porous Media, University of Delaware, Newark, Delaware, July, 1985; Member, Solid Waste Environmental Studies Advisory Committee, Electric Power Research Institute, 1985- .